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Design of a steel-skeleton

Office Building

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DESIGN OF A STEEL-SKELETON
OFFICE BUILDING

BY

PETER WOLFF SEITER

THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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This is to certify that the thesis of PETER WOLFF SEITER entitled Design of a Steel-Skeleton Office Building is approved by me as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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C O N T E N T S .

Page.

Introduction -----	1.
Description of Building -----	3.
Method of Design -----	5.
Art. 1. Determination of Typical Floor-plans, Showing Location of columns, Elevators, Light-Court, and Smoke-Stack.	6.
Art. 2. Arrangement of Columns, Girders, and Floor- beams.	12.
Art. 3. Selection of Fire-proof Floor-arch, and Computation of Weights.	13.
Art. 4. Determination of Height of Building, Thick- ness of Walls Throughout, and Computation of Weights.	15.
Art. 5. Schedule of Columns, and Loads carried by Each.	24.
Art. 6. Design of Floorbeams.	41.
Art. 7. Design of Girders.	44.
Art. 8. Design of Columns.	51.
Art. 9. Design of Wind Bracing.	56.
Art. 10. Design of Spandrel Section.	58.
Conclusion-----	60.



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DESIGN OF A STEEL-SKELETON OFFICE BUILDING.

INTRODUCTION.

The excessive thickness of load-bearing masonry walls in tall building construction made that type of construction uneconomical, and architects began to consider the use of cast-iron columns set in the walls to relieve them of a part of the load. This was the inception of the modern steel building, for it was immediately apparent that, on account of the difference in expansion of the cast-iron columns and the masonry piers, it would be necessary to support the latter at each floor, and thus distribute the movement.

Mr. W. L. Jenny was the originator of this idea (Home Insurance Building, 1883,, and it was developed rapidly, especially in Chicago and New York. In fact, it is said that, except for magnitude and luxuries, the designer of twenty years ago completely solved the problem.

The rapidity of erection, which alone would insure this type of building a permanent place in the building world, is due to the modern organization of engineering forces. Such an organization requires the services of an architect who plans the building; a civil engineer, who designs the steel frame and foundation to carry all the superimposed loads; a mechanical engineer, who designs the power plant needed to run the elevators, generators, pumps, and furnish the steam heat; an electrical engineer to design and install generators and motors which furnish power and light; a

sanitary engineer to lay out the water supply and sewerage disposal plant; and lastly, a fire-protection engineer and superintendent of construction. The work of these men is largely interdependent, and it is well to have them all under one roof; yet, in practice, it is customary for an architectural firm to send their plans out to a structural engineering firm and have them design a suitable steel frame to fit the given plans.

The designer of the steel frame must be in close touch with the architect's ideas, and should strive to keep in harmony with them, even though excessive metal, which is inserted merely for arts sake, is required in certain details. He must also be as thorough in his details as if the metal frame were to be visible instead of being concealed under fire-proofing and plaster. His work is usually done in great haste and under pressure; his reward is the knowledge of having added one more to the already large list of absolutely safe and non-collapsible buildings.

DESCRIPTION OF BUILDING.

Although it has been stated that the design of a steel frame is usually based upon the architect's layout of the floor-plan and elevation of the exterior of the building, it is evident that such procedure cannot be followed out in this design. Hence a typical floor-plan must be assumed, and the following plan was chosen after a study of many successful office buildings.

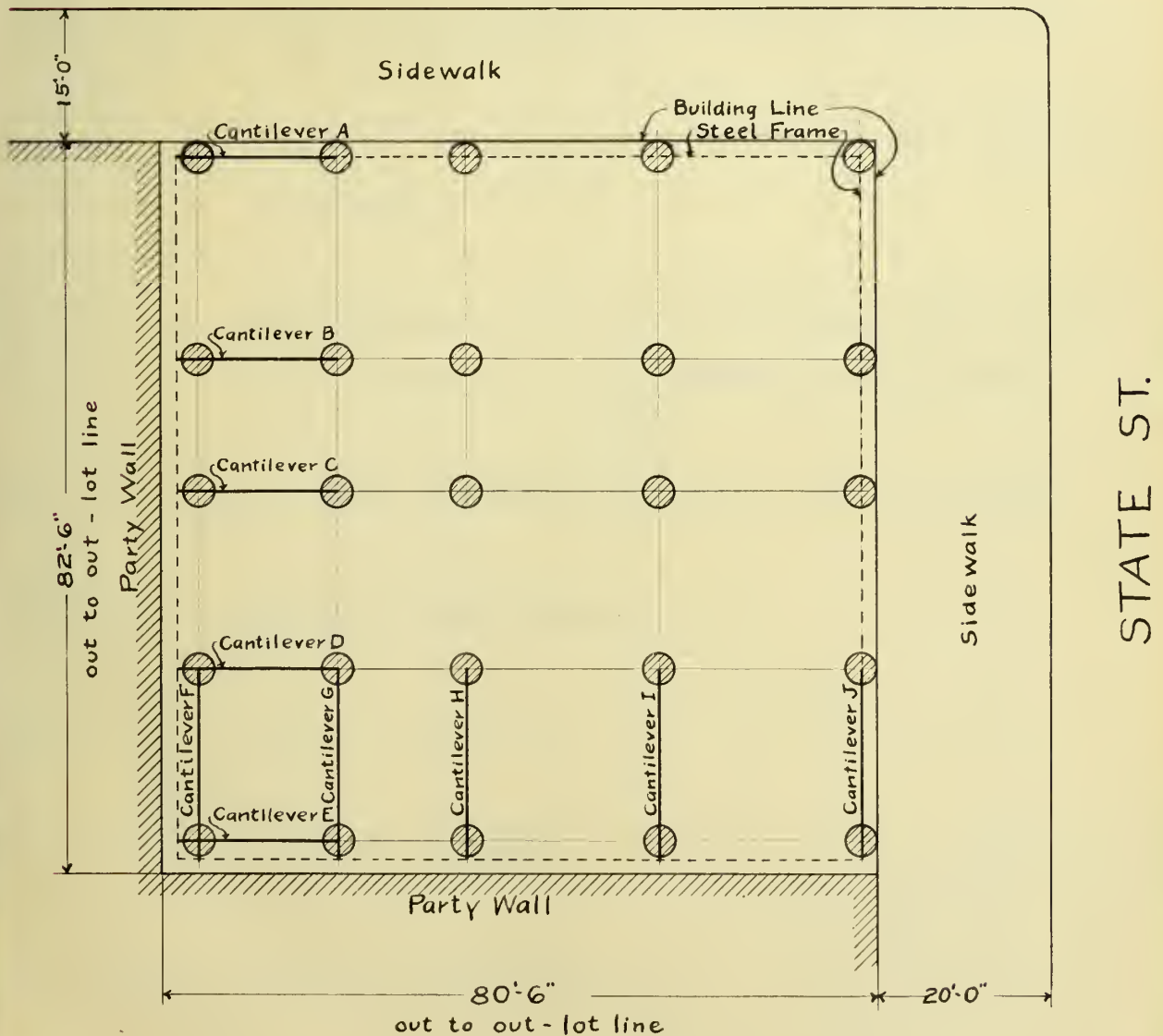
The design to be made in this thesis is that of a 12-story office building: the site adopted as giving good scope for the application of the principles of design is the southwest corner of Adams and State Streets, Chicago. This plot is nearly square, the front on Adams Street being 80-1/2 feet and on State Street, 82-1/2 feet. (See Plate I).

As will be seen on the map of the site, there are no alleys in the rear of the building, hence only the sides facing the street are assured of permanent natural light. The interior of the building will be lighted by the use of a light court above the third floor, opening to the south.

MAP OF SITE

Showing Location of Caissons and Cantilevers

ADAMS ST.



Note- Shaded portions now occupied by 5-story buildings; no alleys.

METHOD OF DESIGN.

The design of the steel frame will be taken up by articles in the following order:

- Art. 1. Determination of Typical Floor-plans, Showing Location of Columns, Elevators, Light-Court, and Smoke Stack.
- Art. 2. Arrangement of Columns, Girders, and Floorbeams.
- Art. 3. Selection of Fire-proof Floor-arch, and Computation of Weights.
- Art. 4. Determination of Height of Building, Thickness of Walls Throughout, and Computation of Weights.
- Art. 5. Schedule of Columns and Loads Carried by Each.
- Art. 6. Design of Floorbeams.
- Art. 7. Design of Girders.
- Art. 8. Design of Columns.
- Art. 9. Design of Wind Bracing.
- Art. 10. Design of Spandrel Section.

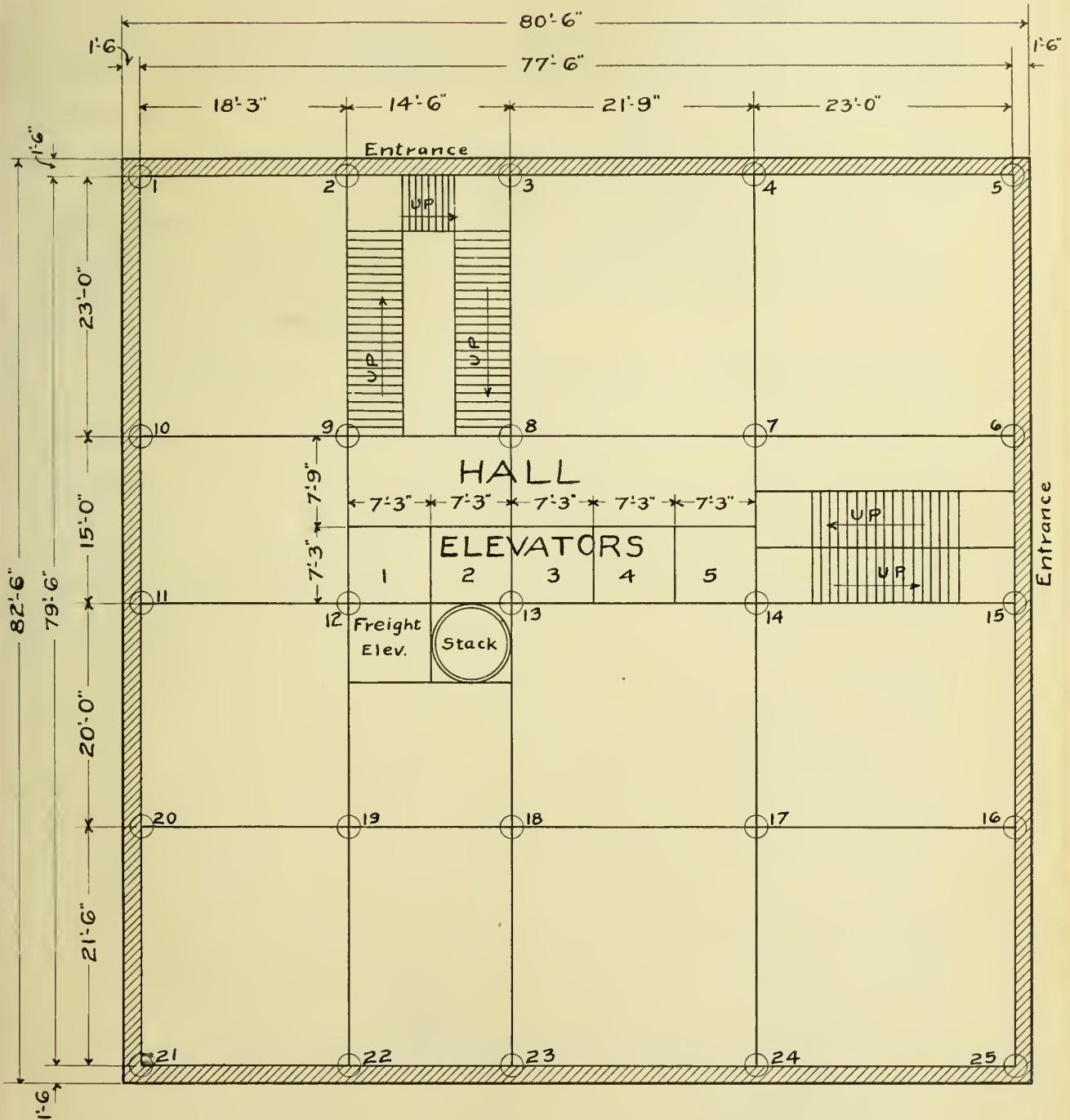
Art. 1. DETERMINATION OF TYPICAL FLOOR-PLANS, SHOWING
LOCATION OF COLUMNS, ELEVATORS, LIGHT-COURT,
AND SMOKE STACK.

As is customary in buildings of this character, the steel frame is set 1-1/2 feet from the property line on all sides, making the plan of the frame 77 feet 6 inches x 79 feet 6 inches, outside dimensions. (See Plate II). The location of the exterior columns is governed by the architectural effect desired, and the location of the interior columns by the convenience and comfort of the future tenants. The standard practice of a large architectural firm in Chicago is to make the maximum panel length 23 feet, thus providing for two offices 11 feet 6 inches above the second floor; and leaving a desirable store front on the main floor.

It is impracticable and even impossible to make comparisons as to the relative economy of many columns with short girders and few columns with long girders, because of local conditions which influence each case. In this building it was considered that the economical design would be to make as few panels as possible without exceeding the above noted panel length of 23 feet, and without causing excessive load concentrations on the footings.

The panels on State Street are made as nearly as possible of the maximum span, leaving a 15-foot entrance. The panels facing Adams Street are governed by the elevator spacing, one panel inclosing two elevators, a distance of 14 feet 6 inches; and another, three elevators, a distance of 21 feet 9 inches. The important corner panel is made 23 feet each way.

(See Plate II).

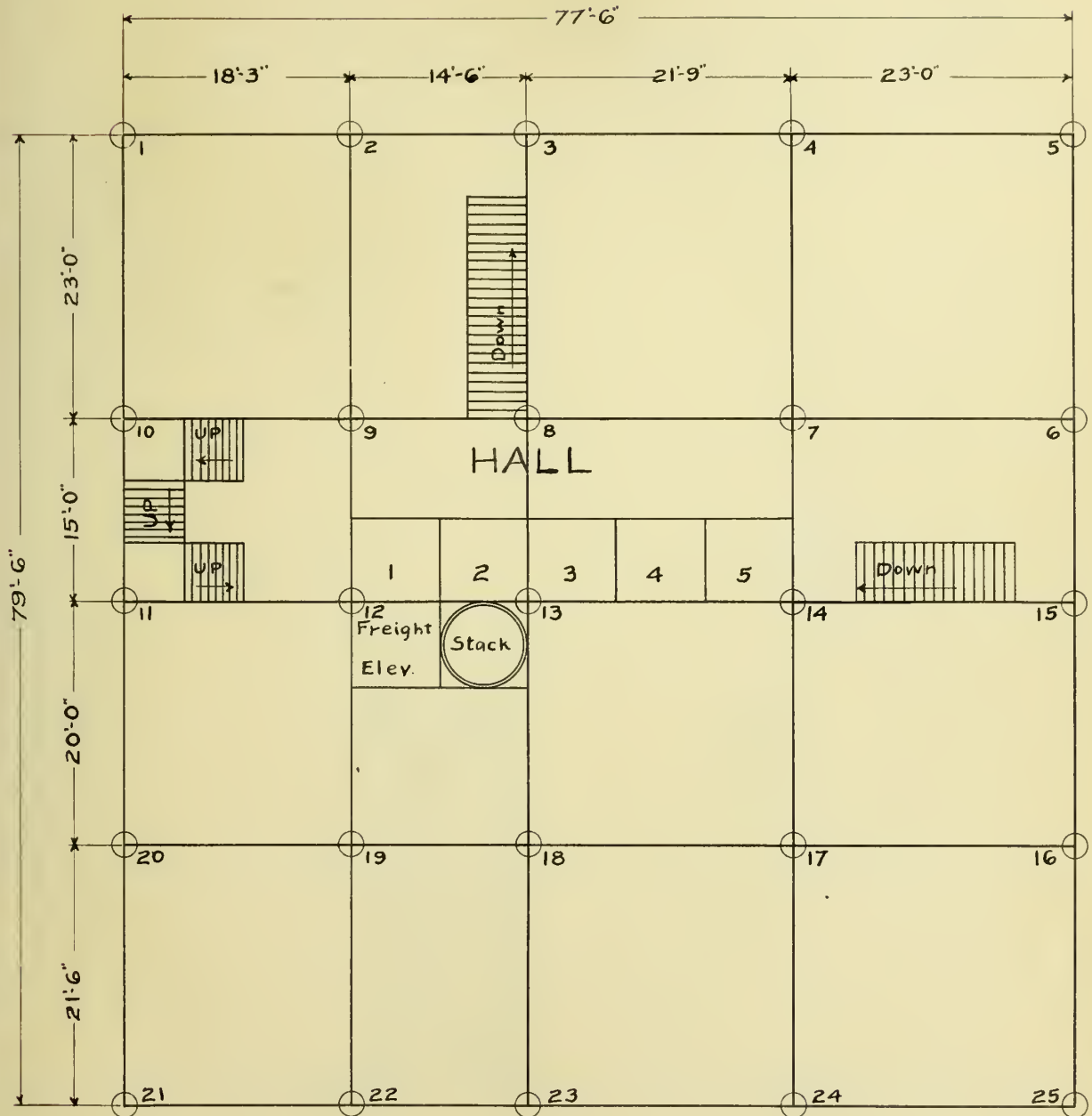


First Floor-Plan.

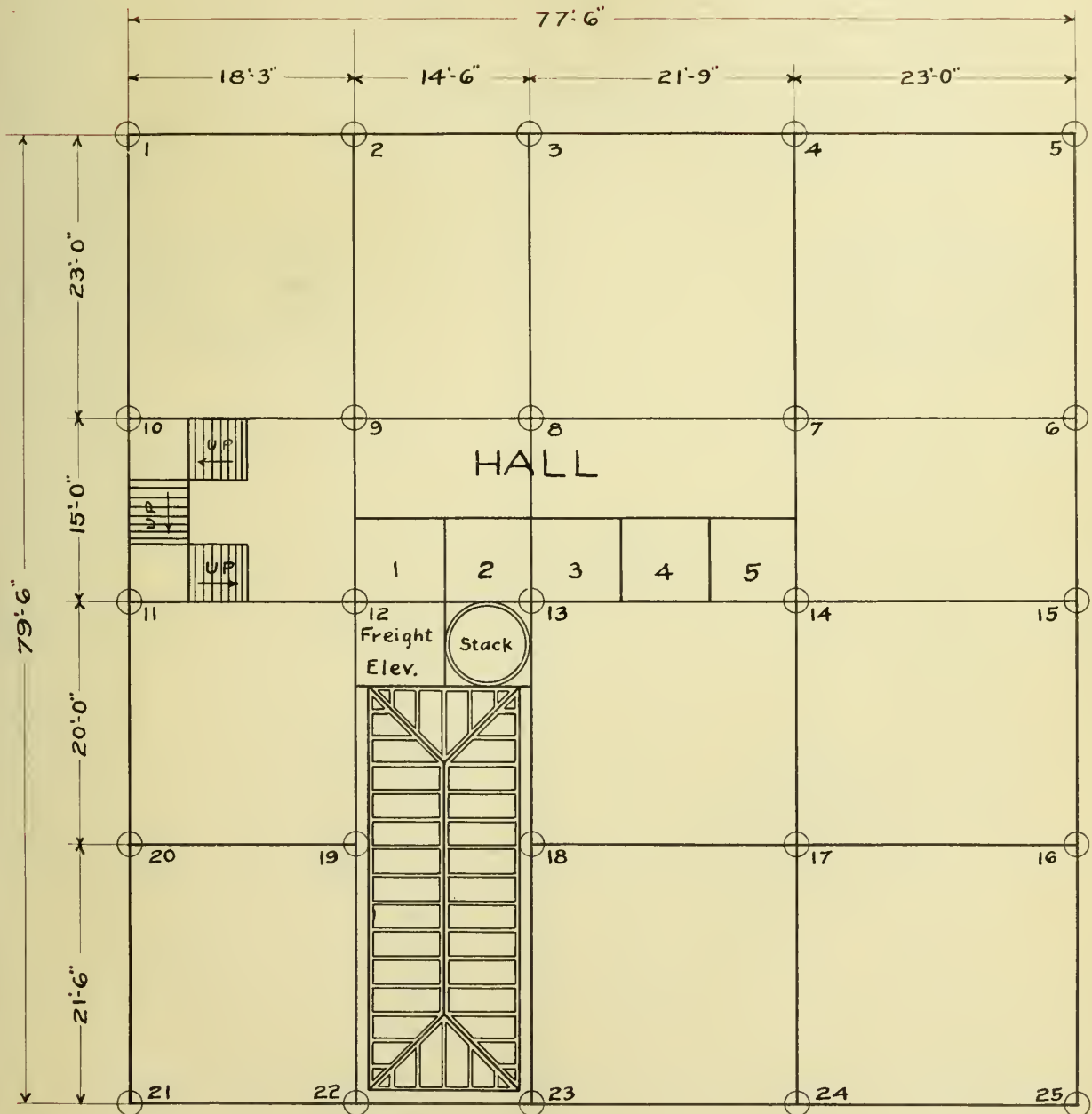
The elevator shafts are placed in the darkest portion of the building, and the stack and freight elevators are placed in panel 12-13-18-19, which is in the light-court above the third floor. This arrangement gives maximum floor space for office use.

The second floor-plan shows the dimensions of the steel frame alone, and gives the location of the main stairs. The stairs above this floor will be regarded as placed in panel 10-11-12. (See Plate III).

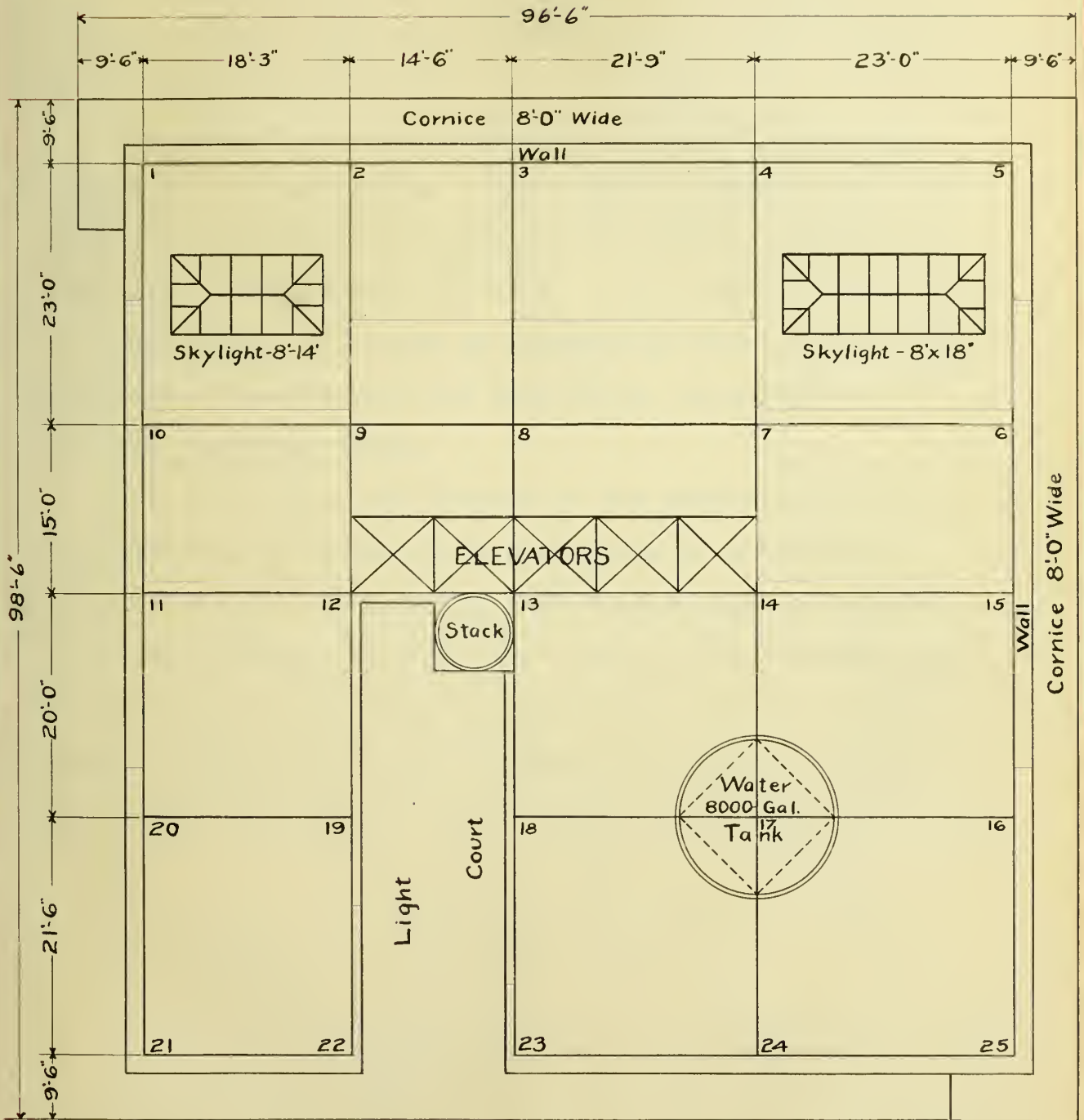
Plate IV shows the third floor-plan, and the bottom of the light-court as indicated by the skylight. The freight elevator ends at this floor, therefore, the light-court will be entirely unobstructed from this floor up to the top of the building (see Plate V) except for the 6-foot stack in the corner.



Second Floor-Plan



Third Floor-Plan



Plan of Roof

Art. 2. ARRANGEMENT OF COLUMNS, GIRDERS, AND FLOOR-BEAMS.

The column centers are determined by the panels, as chosen in Art. 1, and are indicated by the numbers from 1 to 25. (See Plate II).

The girders are now located by connecting the column centers by straight lines. They serve to support the floor-beams and transfer their loads to the columns, and also to brace the columns during erection. The girders connecting the wall columns are known as spandrel-girders, because they support the masonry wall for each story, in addition to carrying the floor loads.

The floor-beams are located in the panels bounded by four girders, but there is no uniformity of practice in their selection. The spacing will depend upon the type of floor-arch chosen, (See Art. 3); but will not exceed 6 feet in any case. If possible, an even number of beams should be used in each panel to avoid loading the girder at its center; but in this design it was not considered practicable to do so.

Art. 3. SELECTION OF FIRE-PROOF FLOOR-ARCH, AND COMPUTATION OF WEIGHTS.

The design of a steel frame for an office building must naturally begin with the selection of a suitable fire-proof floor-arch system, for all the calculations for columns and foundation depend upon the dead and live floor loads.

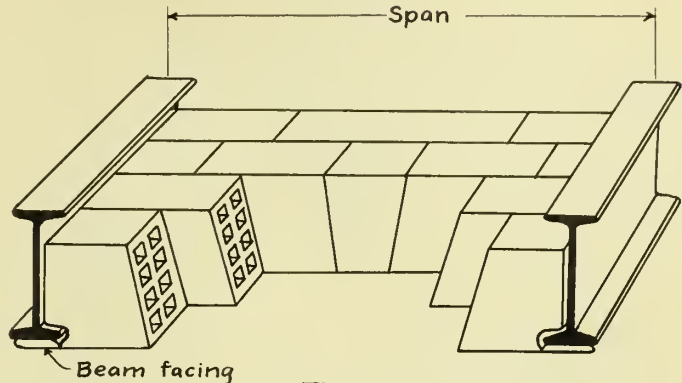


Fig 1.
End-Construction Terra-cotta Arch.

The floor system is especially important in buildings of the curtain-wall type, as it is, to a great extent, relied upon to furnish lateral stiffness to the steel frame. Without entering into a discussion of the various patented floors which are on the market, the "Lee" end-construction terra-cotta arch (Fig. 1), will be used in this design, for the following reasons:

- (a),. it fills the whole depth of the beam, and is well adapted to give lateral rigidity;
- (b),. it gives a flat ceiling;
- (c),. the floor-beams are completely protected from fire;
- (d),. it gives a very low dead weight. (32 pounds per square foot),.

From the tentative floor-framing plan, it is seen that the span of the floorbeams varies from 5 feet to 5 feet 9 inches. For the arch chosen, this span (6 feet and under)

requires a 9-inch depth, and weighs 32 pounds per square foot.

The dead load will be as follows:

Flooring, (7/8-Maple,, 4 pounds per square foot.

Deadening, (cinder concrete), 18 pounds " "

Terra-cotta arch, 32 " " "

Steel beams, 10 " " "

Plastering, 5 " " "

Partitions, 20 " " "

Total,	89	"	"	"
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The live load assumed (standard practice) was 60 pounds per square foot.
Total load on arch = 150 pounds per square foot.

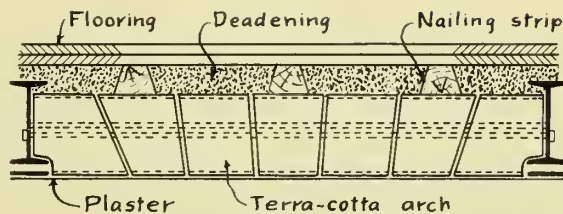


Fig.2.

With this loading on the "Lee" arch, the factor of safety is 6.7. (Freitag - Arch. Eng., Page 94).

Art. 4. DETERMINATION OF HEIGHT OF BUILDING, THICKNESS
OF WALLS THROUGHOUT, AND COMPUTATION OF WEIGHTS.

The height of the building is determined by the standard practice of making the first story 20 feet 8 inches, the second story 15 feet, and all the upper stories 12 feet 6 inches in height. As usual, the sub-basements are extended to a depth of 43 feet below the street level, (see Plate VI).

Height of frame above street =	172'-8"
Depth " " below " =	43'-0"
Total =	215'-8"

The boiler room is set in the rear of the building and at the lowest depth, both to provide room overhead for coal-handling machinery and to permit it to be served by the Subway, which is at that depth in the vicinity of the site chosen. As will be seen on Plate VII, only columns 16 to 25, inclusive, extend to this depth, all the others beginning at the engine room floor, 13 feet higher up.

The walls of the building below the first floor-line are concrete retaining walls, stiffened with I-beams. These are self-supporting, and their weight will not be a part of the loads acting upon the steel frame.

EXTERIOR WALL SECTIONS.

The use of the curtain-wall, supported at each floor-line by spandrel girders, is a characteristic feature of the steel-skeleton building. The wall does not carry any load,

except its own weight, for the first story; but is itself carried by the steel frame, and its chief function is to protect the steel-work from the elements and fire.

It is readily seen that the certain-wall can be of uniform thickness, irrespective of the height of the building, yet conservative architects and the building ordinances require that walls should be of a specified thickness, varying with the height of the building.

The section used in this design will be as follows:-

1st story -	20" walls.
2nd to 6th-	16" walls. (See Plate VIII).
6th to roof-	12" walls.

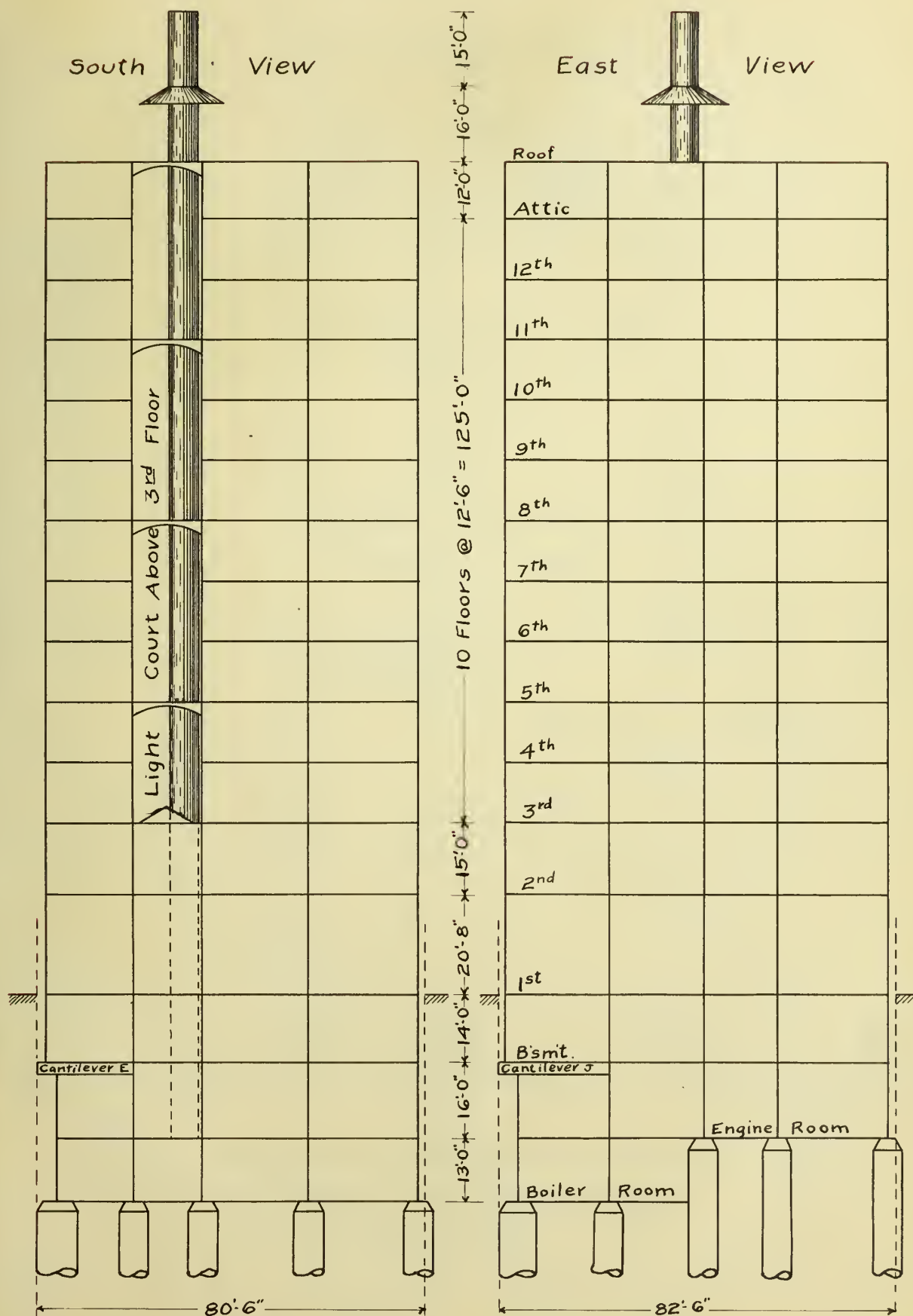
Using a weight for brick and terra-cotta of 112 pounds per cubic foot, the weights of walls are as follows:

12" wall = 112 pounds per square foot of wall area.

16" wall = 149 pounds per square foot of wall area.

20" wall = 187 pounds per square foot of wall area.

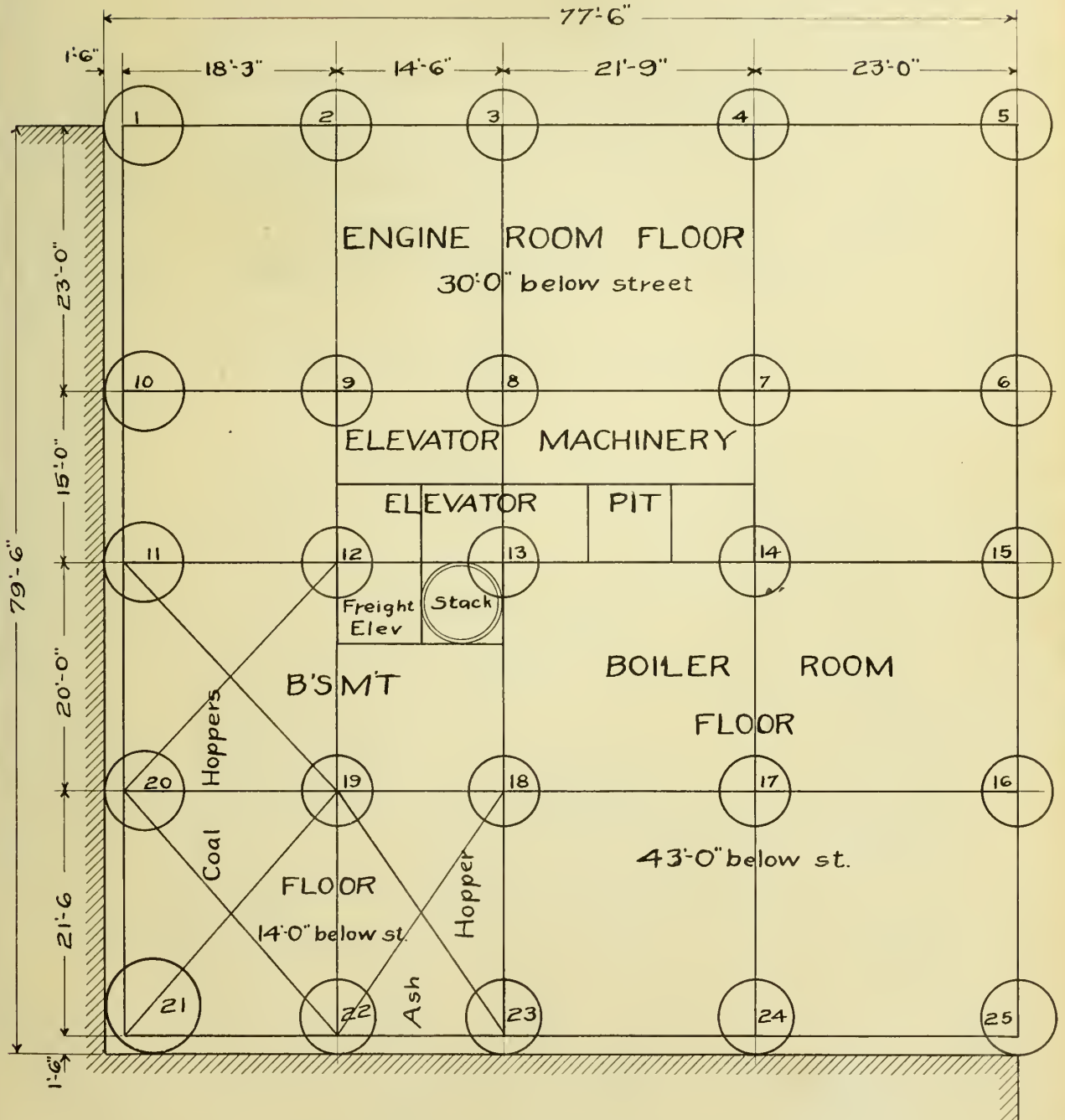
(17)
Diagrammatic Elevation of Steel Frame



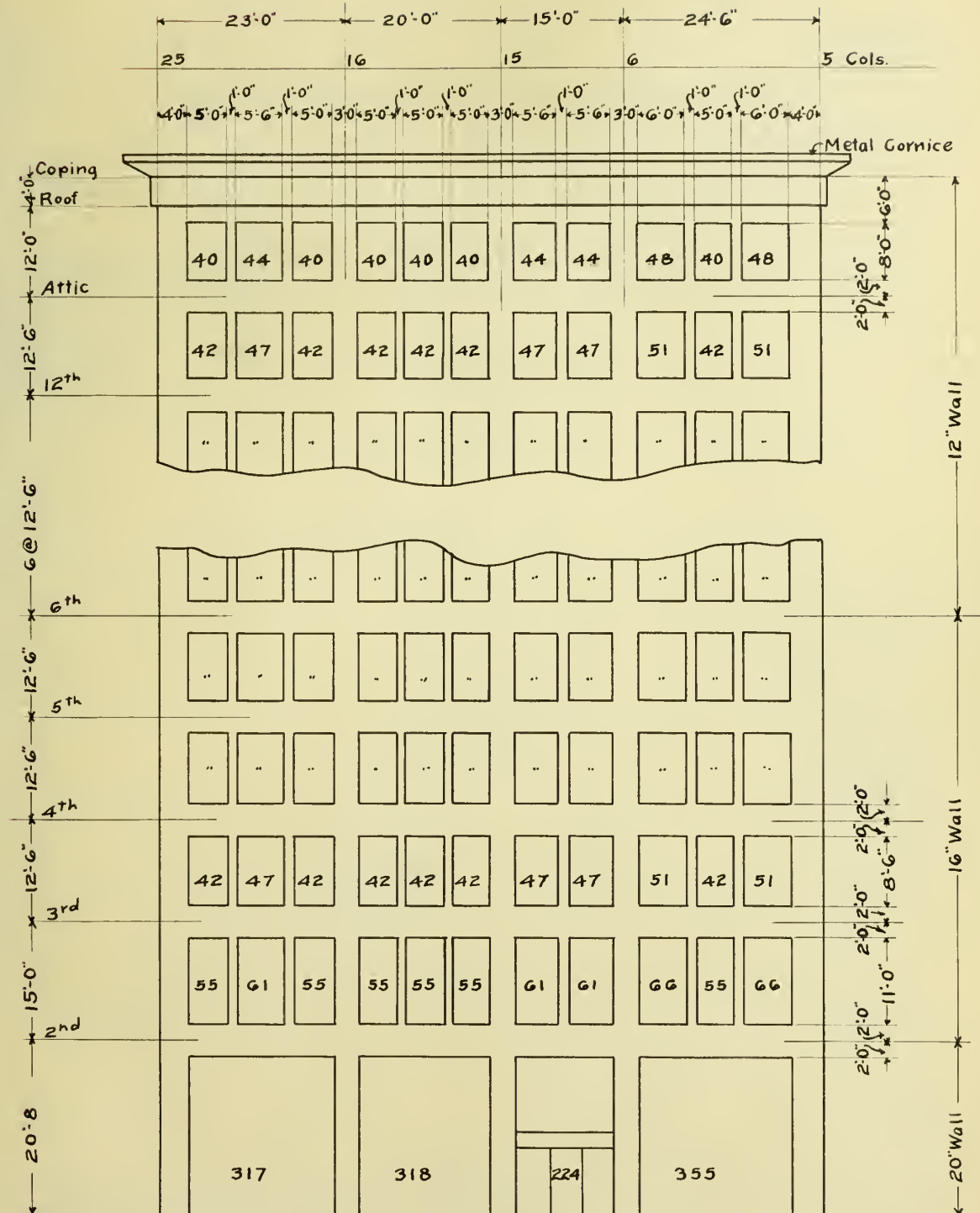
(18)

FOUNDATION PLAN

Showing Caissons & Sub-basmt Floors
with Elevations below Street

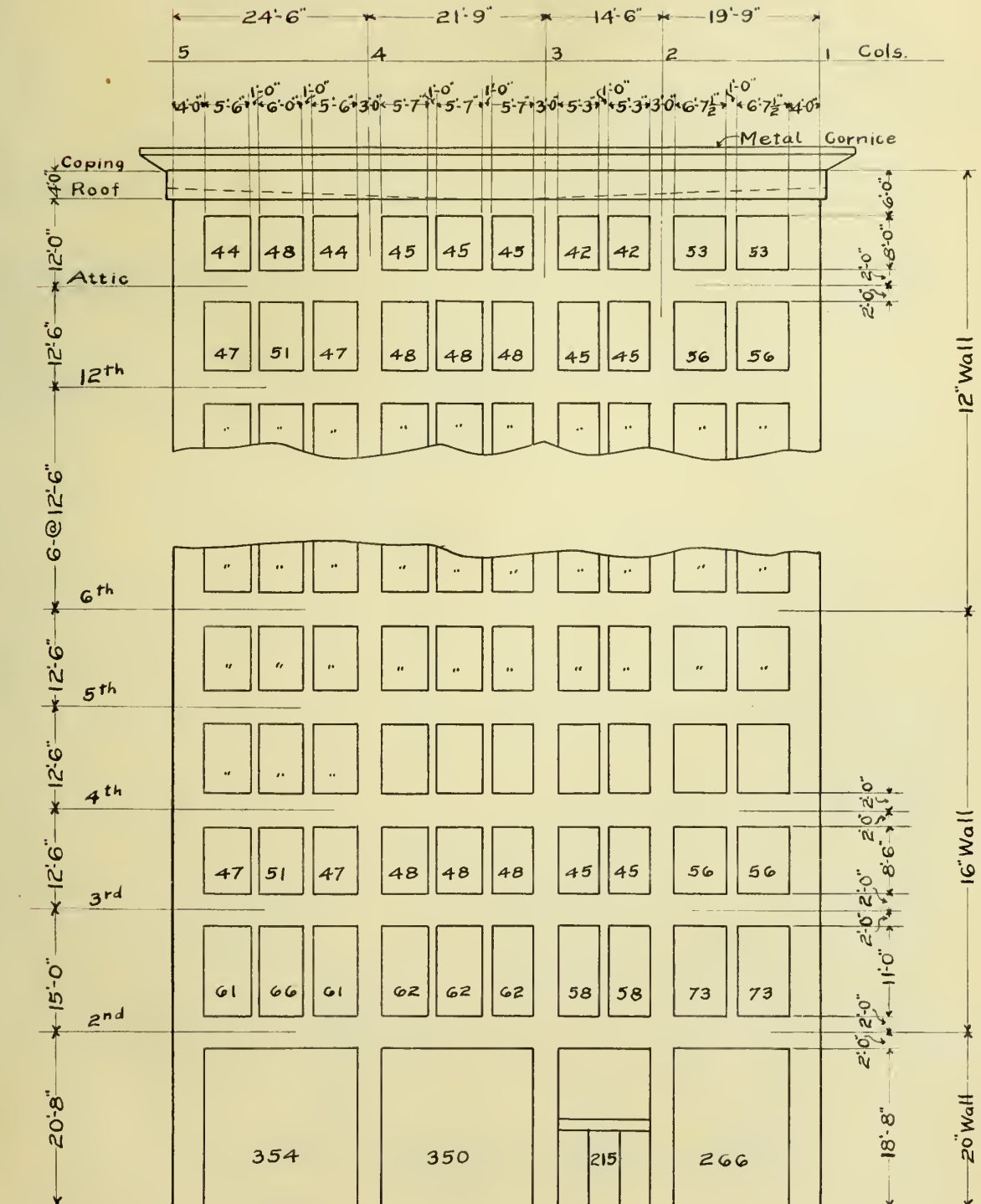


Caissons 1, 10, 11, 20, 21, 22, 23, 24 and 25,
which support cantilever girders, are
tangent to party wall.



Elevation of East Front (State St.)

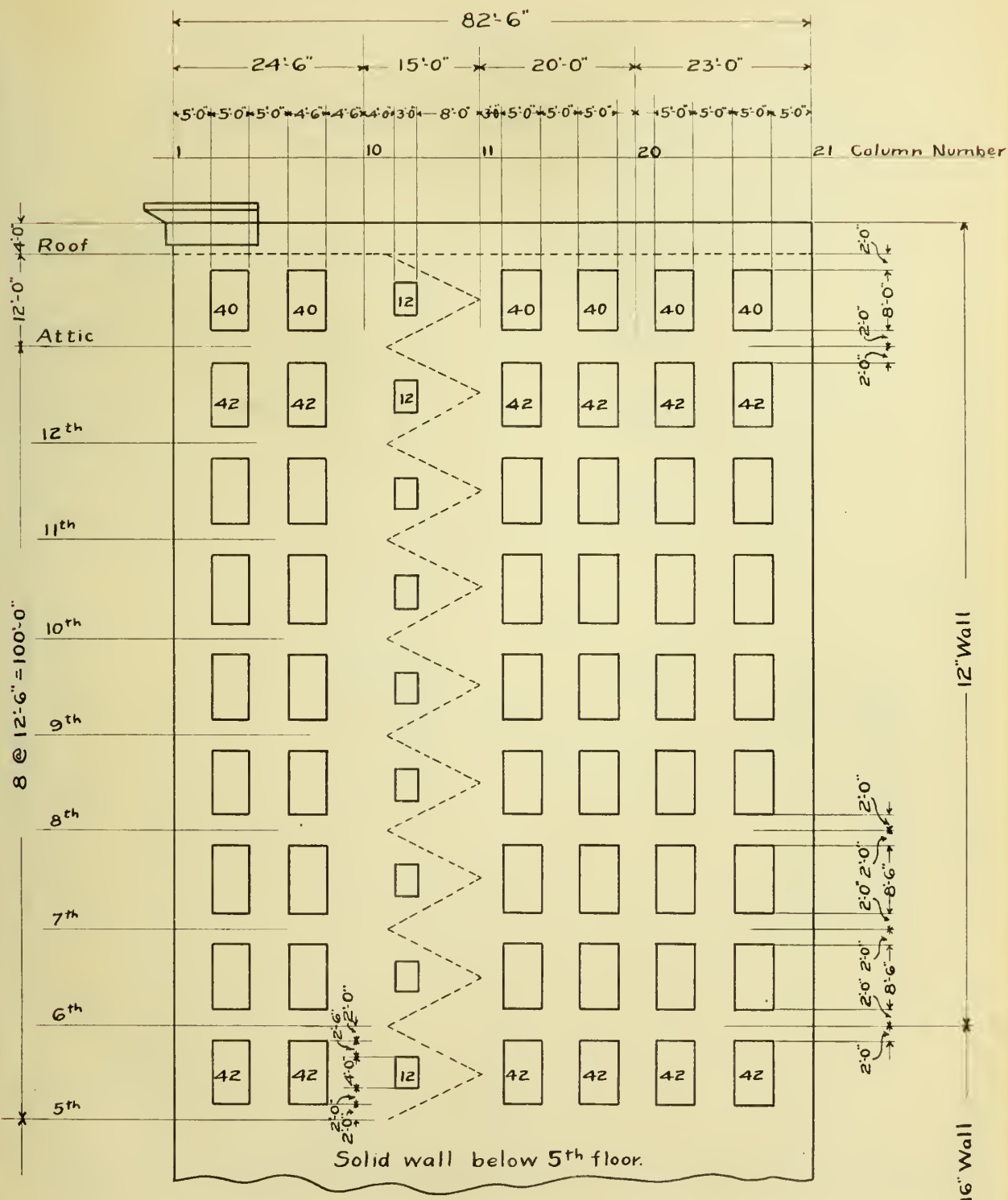
Note- Numbers on windows indicate sq. ft. of area deducted to get spandrel loads.



Elevation of North Front (Adams St.)

Note- Numbers on Windows indicate sq.ft. of area deducted to get spandrel loads.

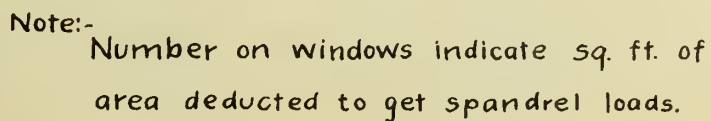
(21)

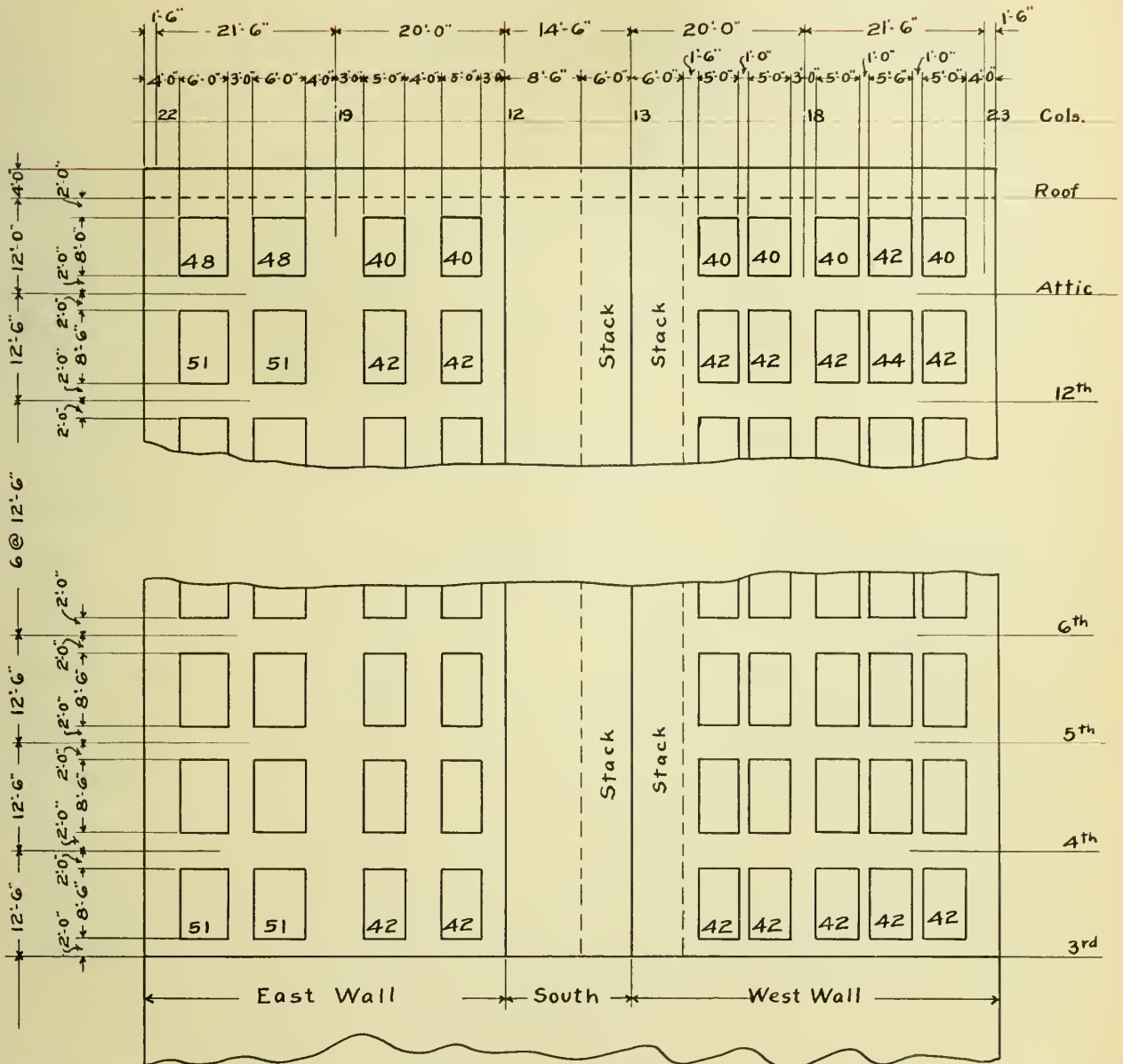


Elevation of West Wall

Note:-

Numbers on windows indicate sq. ft. of area deducted to get spandrel loads.





Development of Light-Court Walls.

All 12-Inch Curtain Walls

Note:-

Numbers on windows indicate sq. ft. of area deducted to get spandrel loads.

Art. 5. SCHEDULE OF COLUMNS AND LOADS CARRIED BY EACH.

The loads on the columns are divided into four classes, as follows:

- (a). dead and live floor loads;
- (b). exterior wall loads;
- (d). wind loads;
- (c). and miscellaneous loads (cornice, elevators, and water tanks).

These loads are determined for each column in the following manner.

(a). The floor area supported by each column is computed from the floor-plan. The dead load on the area is constant (see Art. 3) and is 90 pounds per square foot. The live load varies, being 60 pounds per square foot on the attic and is diminished by 5 per cent for each lower floor until a value of 50 per cent is reached. This value is used for column loads without further reduction for all succeeding floors, except on the first and second floors, where a higher live load is assumed. (See Table 3).

(b). The exterior wall loads are computed by noting the fraction of the total perimeter of the wall which is supported by each column stack. This length is multiplied by the height of the story, giving the area of the wall in question; deducting the window spaces and multiplying the remainder by the thickness of the wall for that story, gives the volume of the brickwork in the required section. In this design, the walls are 20 inches, 16 inches, and 12 inches

thick, as shown on Page 19. The weight of a cubic foot of the wall is taken as 112 pounds.

Example 1.

Column 1. Perimeter = 22.125'

Height of wall from attic to top of coping, = 18' 0"

Gross area of wall, = 22.125 x 18 = 398 sq. ft.

Deduct for windows, = 93 " "

Net Area, = 305 " "

Example 2.

Height of wall from 12th-floor to attic = 12' 6"

Gross area of wall = 22.125 x 12.5 = 277 sq. ft.

Deduct for windows, = 98 " "

Net area, = 179 " "

(See Table V).

(c). Miscellaneous loads.

Weight of cornice assumed as 500 pounds per lineal foot.

Weight of water tank on Column 17.

8,000 gal. = 66,700 pounds.

Tank (3/8" steel) = 3,150 pounds.

69,850 pounds, say 70,000 pounds.

Elevators, 4,000 pounds each.

TABLE 1.

COMPUTATION OF FLOOR AREAS.

With Lengths of Cornice Supported by Columns.

Col.	Gross Area.	Net Area.	Length of Cornice.	Remarks on Areas.
1.	$11-1/2 \times 9-1/8=105$	105	27'-7 1/2"	All Floors.
2.	$11-1/2 \times 16-3/8=188$	188	16'-4 1/2"	" "
3.	$11-1/2 \times 18-1/8=208$	208	16'-1 1/2"	" "
4.	$11-1/2 \times 22-3/8=257$	257	22'-4 1/2"	" "
5.	$11-1/2 \times 11-1/2=132$	132	32'-6"	" "
6.	$11-1/2 \times 19=218$	218	19'-9"	" "
7.	$19 \times 22-3/8=425$	425		" "
8.	$19 \times 18-1/8=345$	345		" "
9.	$19 \times 16-3/8=311$	311		" "
10.	$19 \times 9-1/8=174$	174		" "
11.	$17-1/2 \times 9-1/8=160$	160		" "
12.	$17-1/2 \times 16-3/8=286$	161		Above 3rd. Floor.
12.	$17-1/2 \times 16-3/8=286$	181		Below " "
13.	$17-1/2 \times 18-1/8=318$	114		Above " "
13.	$17-1/2 \times 18-1/8=318$	154		Below " "
14.	$17-1/2 \times 22-3/8=392$	313		All Floors.
15.	$11-1/2 \times 17-1/2=201$	201	17'-6"	" "
16.	$11-1/2 \times 20-3/4=238$	238	20'-9"	" "
17.	$20-3/4 \times 22-3/8=464$	464		" "
18.	$20-3/4 \times 18-1/8=376$	226		Above 3rd. Floor.
18.	$20-3/4 \times 18-1/8=376$	376		Below " "
19.	$20-3/4 \times 16-3/8=340$	190		Above " "
19.	$20-3/4 \times 16-3/8=340$	340		Below " "

TABLE 1. (Continued).

COMPUTATION OF FLOOR AREAS.

With Lengths of Cornice Supported by Columns.

Col.	Gross Area.	Net Area.	Length of Cornice.	Remarks on Areas.
20.	$20-3/4 \times 9-1/8=190$	190		All Floors.
21.	$10-3/4 \times 9-1/8=98$	98		" "
22.	$10-3/4 \times 10-3/8=170$	98		Above 3rd. Floor.
22.	$10-3/4 \times 16-3/8=176$	176		Below " "
23.	$10-3/4 \times 18-1/8=195$	117		Above " "
23.	$10-3/4 \times 18-1/8=195$	195		Below " "
24.	$10-3/4 \times 22-3/8=240$	240		All Floors.
25.	$10-3/4 \times 11-1/2=124$	124	29'-3"	" "

Total, 6,169 5305 Diff. = 864 square feet.

Area deducted = 5 elevators @ $(7-1/4)^2$ = 263

Light Court, $14.5 \times (21.5 + 20) = \underline{602}$

Check.

865

TABLE 2.
SCHEDULE OF TOTAL FLOOR LOADS.

Column.	1.	2.	3.	4.	5.
	^a	^a	^a	^a	^a
Extra Load.	13.6	8.2	9.1	11.3	16.2
Area sq. ft.	105.0	188.0	208.0	257.0	132.0
Pounds per sq. ft.					
Roof. 58	6.1	10.9	12.1	14.9	7.6
Attic. 131	13.8	24.0	27.2	33.7	17.3
12 129	13.5	24.2	26.8	33.1	17.0
11 127	13.3	23.9	26.4	32.6	16.8
10 126	13.2	23.7	26.2	32.4	16.6
9 124	13.0	23.3	25.8	31.8	16.4
8 122	12.8	23.0	25.4	31.4	16.1
7 120	12.6	22.6	25.0	30.8	15.8
6 118	12.4	22.2	24.6	30.3	15.6
5 117	12.3	22.0	24.4	30.0	15.4
4 115	12.1	21.6	24.0	29.6	15.2
3 113	11.9	21.2	23.5	29.0	14.9
2 125	13.1	23.5	26.0	32.1	16.5
1 155	16.3	29.2	32.2	39.8	20.4
Bsmt. 300	31.5	56.0	62.0	77.0	39.6
Total	221.5	380.6	421.2	519.8	277.4

Note - (a) Cornice Loads.

(b) Elevator.

TABLE 2. (Continued)⁽²⁹⁾

SCHEDULE OF TOTAL FLOOR LOADS.

Column.	6.	7.	8.	9.	10.
Extra Load.					
Area. sq. ft.	9.5 ^a				
Pounds per sq. ft.	218.0	425.0	345.0	311.0	174.0
Roof. 58	12.7	24.6	20.0	18.0	10.1
Attic. 131	28.6	55.7	45.2	40.8	22.8
12 124	28.3	55.0	44.5	40.2	22.5
11 127	27.7	54.0	45.8	39.5	22.1
10 126	27.5	53.5	43.5	39.2	21.9
9 124	27.0	52.7	42.8	38.6	21.6
8 122	26.6	52.0	42.1	38.0	21.2
7 120	26.2	51.0	41.4	37.4	20.9
6 118	25.7	50.0	40.7	36.7	20.5
5 117	25.5	49.7	40.4	36.4	20.4
4 115	25.0	49.9	39.7	35.8	20.0
3 113	24.6	48.0	39.0	35.2	19.7
2 125	27.2	53.0	43.1	38.9	21.8
1 155	35.8	66.0	53.5	48.2	27.0
Bsmt. 300	66.5	127.5	103.5	93.4	52.3
Total,	442.4	841.7	663.2	606.3	344.8

Note - (a) Cornice Loads.

(b) Elevator.

TABLE 2. (Continued).

SCHEDULE OF TOTAL FLOOR LOADS.

Column.	11.	12.	13.	14.	15.
Extra Load.		2.0 ^u	5.0 ^u	3.0 ^b	8.7 ^a
Area.sq.ft.	100.0	161.0	114.0	313.0	201.0
Pounds per sq. ft.					
Roof.	9.3	9.4	6.6	18.1	11.7
Attic.	21.0	21.1	14.9	41.0	26.4
12	20.7	20.8	14.7	40.4	26.0
11	20.3	20.4	14.5	39.8	25.6
10.	20.2	20.3	14.3	39.4	25.3
9.	19.9	20.0	14.1	38.8	25.0
8.	19.5	19.6	13.9	38.2	24.6
7.	19.2	19.3	13.7	37.6	24.1
6.	18.9	19.0	13.5	37.0	23.7
5.	18.7	18.8	13.3	36.6	23.5
4.	18.4	18.5	13.1	36.0	23.1
3.	18.1	181.0 20.4	134.0 15.3	35.4	22.7
2.	20.0	22.6	16.8	39.1	25.1
1.	24.8	28.0	20.8	48.5	31.2
Bsmt.	48.0	54.3	40.2	94.0	60.3
Total.	317.0	334.5	244.7	622.9	407.0

Note-

(a) Cornice load.

(b) Elevators.

(c) Water Tank.

TABLE 2. (Continued).

SCHEDULE OF TOTAL FLOOR LOADS.

Column.	16. a	17. c	18.	19.	20.
Extra Load.	10.3	70.0			
Area sq.ft.	238.0	464.0	226.0	190.0	190.0
Pounds per sq. ft.					
Roof.	13.8	26.9	13.1	11.0	11.9
Attic.	31.2	60.7	29.6	24.9	24.9
12.	30.7	59.8	29.2	24.5	24.5
11.	30.2	59.0	28.7	24.1	24.1
10.	30.0	58.5	28.5	24.0	24.0
9.	29.5	57.5	28.0	23.6	23.6
8.	29.0	56.6	27.6	23.2	23.2
7.	28.6	55.7	27.1	22.8	22.8
6.	28.1	54.7	26.7	22.4	22.4
5.	27.8	54.3	26.4	22.2	22.2
4.	27.4	53.4	26.0	21.8	21.8
3.	26.9	52.5	376.0 42.5	340.0 38.4	21.5
2.	29.8	58.0	47.0	42.5	23.8
1.	36.9	72.0	58.3	52.7	29.5
Bsmt.	71.5	139.0	113.0	102.0	57.0
Total,	443.7	988.6	551.7	408.1	331.3

Note-

(a) Cornice load.

(b) Elevators.

(c) Water tank.

TABLE 2. (Continued),.

SCHEDULE OF TOTAL FLOOR LOADS.

Column.	21.	22.	23.	24.	25. ^a
Extra Load.					14.6
Area sq. ft.	98.0	98.0	117.0	240.0	124.0
Pounds per sq. ft.					
Roof.	5.7	5.7	6.8	13.9	7.2
Attic.	12.8	12.8	15.3	31.5	16.2
12.	12.6	12.8	15.1	31.0	16.0
11.	12.4	12.4	14.8	30.5	15.7
10.	12.3	12.3	14.7	30.2	15.6
9.	12.2	12.2	14.5	29.8	15.4
8.	12.0	12.0	14.3	29.3	15.1
7.	11.8	11.8	14.0	28.8	14.9
6.	11.6	11.6	13.8	28.3	14.6
5.	11.5	11.5	13.7	28.1	14.5
4.	11.3	11.3	13.4	27.6	14.2
3.	11.1	176.0 19.9	195.0 22.0	27.1	14.0
2.	12.3	22.0	24.4	30.0	15.5
1.	15.2	27.3	30.2	37.2	19.2
Bsmt.	29.4	52.8	58.5	72.0	37.2
Total,	194.2	248.2	285.5	475.3	260.0

Note-

(a) Cornice load.

(b) Elevators.

(c) Water tank.

TABLE 3.

UNIT FLOOR LOADS.

Floors.	Loads.	Beams. 100% Live.	Girders. 80% Live.	Columns. 60% Live.
(a)	Dead.	40	40	40
	Live.	30	24	18
Roof. 100.	Total.	70	64	58
	Dead.	95	95	95
	Live.	60	48	36
Attic. 100	Total.	155	143	131
	Dead.	95	95	95
	Live.	60	48	34
12. 95	Total.	155	143	129
	Dead.	95	95	95
	Live.	60	48	32
11. 90	Total.	155	143	127
	Dead.	95	95	95
10. 85	Total.	155	143	126
	Dead.	95	95	95
	Live.	60	48	29
9. 80	Total.	155	143	124
	Dead.	95	95	95
	Live.	60	48	27
8. 75	Total.	155	143	122
	Dead.	95	95	95
	Live.	60	48	25
7. 70	Total.	155	143	120

TABLE 3. (Continued).

UNIT FLOOR LOADS.

Floors.		Loads.	Beams. 100% Live.	Girders. 80% Live.	Columns. 60% Live.
		Dead.	95	95	95
		Live.	60	48	23
6.	65	Total.	155	143	118
		Dead.	95	95	95
		Live.	60	48	22
5.	60	Total.	155	143	117
		Dead.	95	95	95
		Live.	60	48	20
4.	55	Total.	155	143	115
		Dead.	95	95	95
		Live.	60	48	18
3.	50	Total.	155	143	113
		Dead.	95	95	95
		Live.	100	80	30
2.	50	Total.	195	175	125
		Dead.	95	95	95
		Live.	100	80	30
1.	100	Total.	195	175	155
		Dead.	120	120	120
		Live.	300	240	180
Bsmt.	100	Total.	420	360	300

Note -(a) Percent of live load used for columns.

TABLE 4.

SCHEDULE OF EXTERIOR WALL LOADS.

		12" Wall.		16" Wall.		20" Wall.			
Wt. per sq.ft.		112 #		149 #		187 #			
Col.		Attic	6th	5th	4th	3rd	2nd	Total.	
		Attic.to 12	to 5th	to 4.	to 3.	to 2.	to 1st.		
1.	Area.	305	179	179	221	221	259	281	
	Kips.	34.2	20.0	20.7	33.0	33.0	38.6	52.5	358.0
2	Area.	200	105	105	105	105	115	56	
	Kips.	22.4	11.8	15.6	15.6	15.6	17.2	10.5	179.5
3.	Area.	217	109	109	109	109	121	56	
	Kips.	24.4	12.2	16.3	16.3	16.3	18.0	10.5	187.2
4.	Area.	268	136	136	136	136	148	56	
	Kips.	30.0	15.2	20.2	20.2	20.2	22.0	10.5	220.5
5.	Area.	304	161	161	161	161	181	122	
	Kips.	34.0	18.9	24.0	24.9	24.9	27.0	22.8	281.8
6.	Area.	230	119	119	119	119	131	56	
	Kips.	25.8	13.4	17.7	17.7	17.7	19.5	10.5	202.7
10.	Area.	290	184	184	238	238	285	355	
	Kips.	32.5	20.6	27.4	30.5	35.5	42.5	66.5	384.1
11.	Area.	275	177	177	219	219	263	328	
	Kips.	30.8	19.8	26.4	32.6	32.6	39.2	61.5	362.7
15.	Area.	211	109	109	109	109	119	56	
	Kips.	23.6	12.2	16.3	16.3	16.3	17.7	10.5	186.1
16.	Area.	252	152	152	152	152	144	56	
	Kips.	28.2	14.8	19.7	19.7	19.7	21.4	10.5	222.8

TABLE 4. (Continued).

SCHEDULE OF EXTERIOR WALL LOADS.

		12" Wall.		16" Wall.		20" Wall.			
Wt. per sq.ft.		112 #		149 #		187 #			
Col.		Attic.	6th	5th	4th	3rd	2nd		
		to 12.	to 5.	to 4.	to 3.	to 2.	to 1st.	Total.	
20.	Area.	294	176	176	260	260	312	388	
	Kips.	35.0	19.7	26.2	30.8	38.8	46.5	72.5	393.7
21.	Area.	344	225	225	267	267	320	400	
	Kips.	30.6	25.2	33.6	39.8	39.8	47.8	75.9	451.0
22.	Area.	336	216	216	216	216	246	306	
	Kips.	37.6	24.2	32.2	32.2	32.2	36.6	57.3	397.5
19.	Area.	286	167	167	167	125	0	0	
	Kips.	32.0	18.7	24.9	24.9	18.6	0	0	231.3
12.	Area.	244	155	155	155	124	0	0	
	Kips.	27.4	17.4	23.1	23.1	18.5	0	0	213.9
13.	Area.	264	176	176	176	145	0	0	
	Kips.	29.6	19.7	26.2	26.2	21.0	0	0	241.5
18.	Area.	253	133	133	133	91	0	0	
	Kips.	28.4	14.9	19.8	19.8	13.6	0	0	185.9
23.	Area.	352	225	223	225	223	212	340	
	Kips.	39.4	25.0	33.2	35.2	33.2	40.5	63.5	388.5
24.	Area.	383	260	260	260	280	335	420	
	Kips.	43.0	29.1	38.8	38.8	41.8	50.0	78.5	494.6
25.	Area.	365	232	232	232	232	271	122	
	Kips.	41.0	26.0	34.6	34.6	34.6	40.5	22.8	390.1

TABLE 5.

TOTAL REACTIONS AT FOOTINGS.

Col.	Floor Load.	Wall Load.	Steel Frame. ^x	Total.
1.	221.5	356.0	87.0	664.5
2.	380.6	179.5	84.0	644.1
3.	421.2	187.2	91.5	699.9
4.	519.8	229.5	112.5	861.8
5.	277.4	261.8	84.0	643.2
6.	442.4	202.7	96.8	741.9
7.	841.7	0	126.0	967.7
8.	663.2	0	99.5	762.7
9.	606.3	0	91.0	697.3
10.	344.8	384.1	109.5	838.4
11.	317.0	362.7	102.0	781.7
12.	334.5	213.9	82.0	630.4
13.	244.7	241.5	73.0	559.2
14.	522.9	0	94.5	717.4
15.	407.0	186.1	89.0	682.1
16.	443.7	222.8	100.0	766.5
17.	988.6	0	148.0	1136.6
18.	551.7	185.9	110.5	848.1
19.	408.1	231.3	96.0	735.4
20.	331.3	393.7	108.6	833.6
21.	194.2	451.0	96.7	741.9
22.	248.2	397.5	97.0	742.7
23.	285.5	388.5	101.0	775.0
24.	475.3	494.6	145.4	1115.3
25.	260.0	390.1	97.6	747.7

1083.6

5982.4

2523.1

x Estimated at 15 percent of Floor and Wall Loads.

WIND LOADS ON COLUMNS.

Wind load = 30 lbs. per sq. ft.

Height of building exposed =

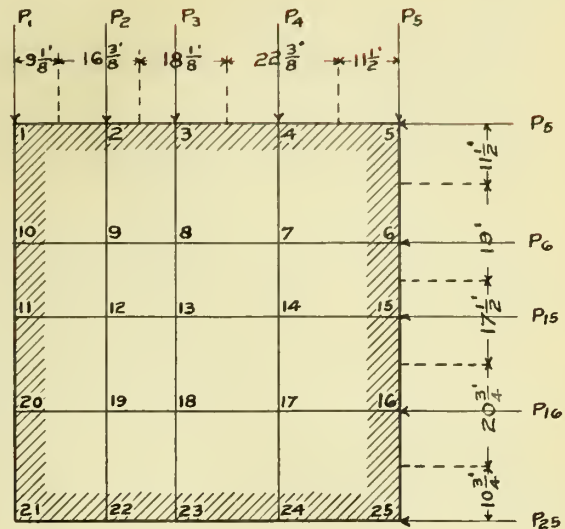
172'-8".

Point of application of

resultant wind = $\frac{2}{3}$ of

height above ground =

$\frac{2}{3} \times 172'-8" = 115.1'$



$P = \text{Panel} \times 172'-2/3 \times 30 \text{ lbs.}$

1 =	47,300 #
2 =	85,000 #
3 =	94,000 #
4 =	116,000 #
5 =	59,000 #
6 =	98,600 #
15 =	90,800 #
16 =	107,500 #
25 =	55,800 #

Note -

Column loads taken
from Table 5.

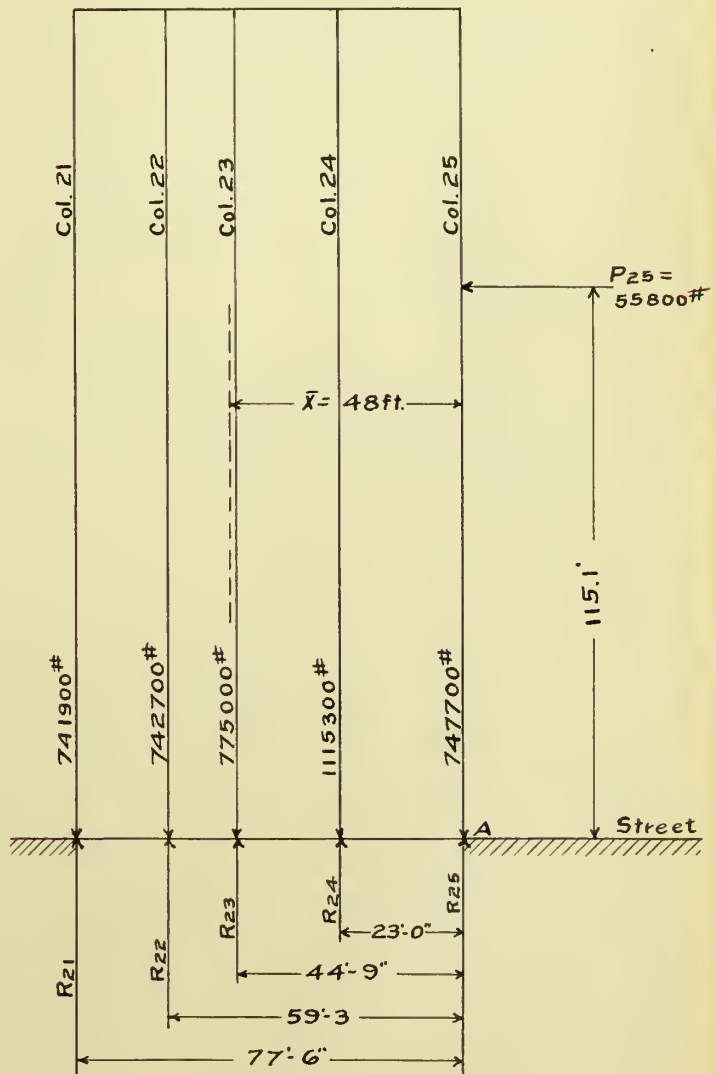


Fig. 3.

Considering the building as a vertical, cantilever beam, it is found that the dead load on the footings is so great (19,337,000 pounds) in comparison with the horizontal wind pressure, (428,000 pounds) that no negative reaction occurs on the windward edge.

For the purpose of analysis, the columns in the rear walls will be considered. (See Fig. 3, Page 38). The wind load (P_{25}) = $10 \frac{3}{4} \times 172 \frac{2}{3} \times 30 = 55,800$ pounds. The vertical column loads shown are the combined floor, wall, and steel frame loads. The resultant of these loads, taking moments about A, is

$$\bar{R}x = 741.9 \times 77.5 + 742.7 \times 59.25 + 775.9 \times 44.75 + 1115.3 \times 23$$

$$R = 741.9 + 742.7 + 1115.3 = 3374.9 \quad \bar{x} = 48 \text{ feet.}$$

Let E = equilibrant acting opposite to R.

Then taking moments of all forces about A,

$$-55,800 \times 115.1 - R\bar{x} + E\bar{x} = 0.$$

$$E = \frac{55,800 \times 115.1}{48} + 3375.1 = 3509.$$

Assuming wind reaction of each column in direct proportion to its dead load:

$$R_{21} = (E/R)741.9 = 773$$

$$R_{22} = (E/R)742.7 = 775$$

$$R_{23} = (E/R)775.0 = 806$$

$$R_{24} = (E/R)1115 = 1160$$

$$R_{25} = (E/R)747.7 = 778$$

Example 2. Consider Panels at P_{16} .

$$P_{16} = 107,500 \text{ pounds.}$$

$$R = 833.6 + 735.4 + 848.1 + 1136.6 = 3553.7$$

$$\bar{x} = \frac{833.6 \times 77.5 + 735.4 \times 59.25 + 848.1 \times 44.75 + 1136.6 \times 23}{3553.7}$$

$$= 48.5 \text{ feet.}$$

Moments of all forces about A.

$$-P_{16} \times 115.1 - R\bar{x} + E\bar{x} = 0. \quad E = \frac{107.6 \times 115.1 + 3558.7}{48.5} = 3808.$$

As before,

$$R_{20} = (E/R) 833.6 = (3808/3553) 833.6 = 895.9$$

$$R_{19} = (E/R) 753.4 = \quad \quad \quad = 790.0$$

$$R_{18} = (E/R) 848.1 = \quad \quad \quad = 910.0$$

$$R_{17} = (E/R) 1136.6 = \quad \quad \quad = 1220.0$$

$$R_{16} = (E/R) 766.5 = \quad \quad \quad = 822.0$$

The percent increase in the reaction at base of column 1,

$$\text{is } \frac{1220 - 1136.7}{1136.7} = 0.073$$

$$\text{For } R_{20}, \text{ percent increase} = \frac{895 - 833.6}{833.6} = 0.073, \text{ etc.}$$

Since the increase of the reaction due to the wind is less than 1 percent, when the wind is taken at its maximum value, and considered as acting over the whole surface of the building, (which is not likely to occur since the building is protected for a height of several stories), the effect of the wind will be neglected in figuring the vertical reactions.

Art. 6. DESIGN OF FLOOR BEAMS.

Dead Floor Load = 95 lbs. per sq. ft.

Live " " = 60 " " " "

Total " " = 155 " " " "

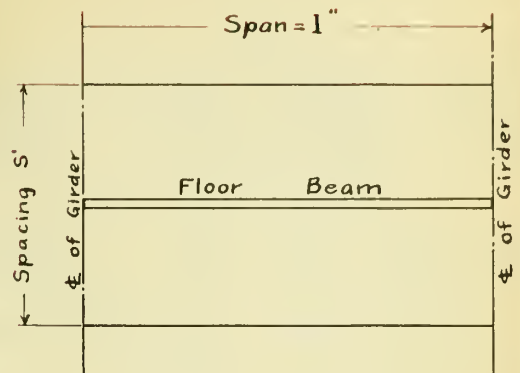
 $M = 1/8 w l^2$, where $w = 155 \times S$.

Fig. 4.

Example - Uniform loading.

Panel 1. (See Plate 13).

Span $l = 18'-3'' = 214''$ Beams spaced $S = 5'-9'' = 5.75'$ $M = 1/8 w l^2 = 1/8 (155 \times 5.75) (219)^2 =$

535,000 lb. inches.

Allowable stress = 16,000 lb. inches.

 $M = S I/c \quad I/c = M/S = \frac{535,000}{16,000} = 33.4$ Use $12'' \times 31 \frac{1}{2}''$ I, section modulus = 30

Example. Concentrated loading.

Panel 6 & 7. Between Elevators.

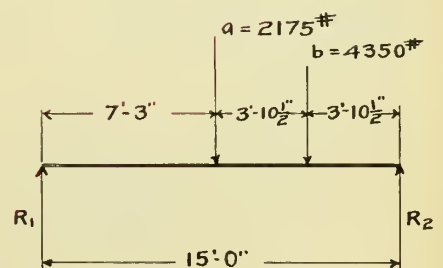
Load $b = (3'-10 \frac{1}{2}'' \times 7'-3'') 155 = 4,350\#$ Load $a = \frac{(3'-10 \frac{1}{2}'' \times 7'-3'') 155}{2} = 2,175\#$ $R_1 = \frac{2,175 \times 7.75}{15} + \frac{4,350 \times 3.88}{15} = 2,260\#$ $M_b = R_1 \times 11.13 = 2,175 \times 3.88 = 16,450$ lb. ft. Max. Moment. $M_a = R_1 \times 7.25 = 16,350$ " " $I/c = \frac{16,450 \times 12}{16,000} = 12.3$ Use $8'' \times 18\#$ I. Section modulus = 14.2

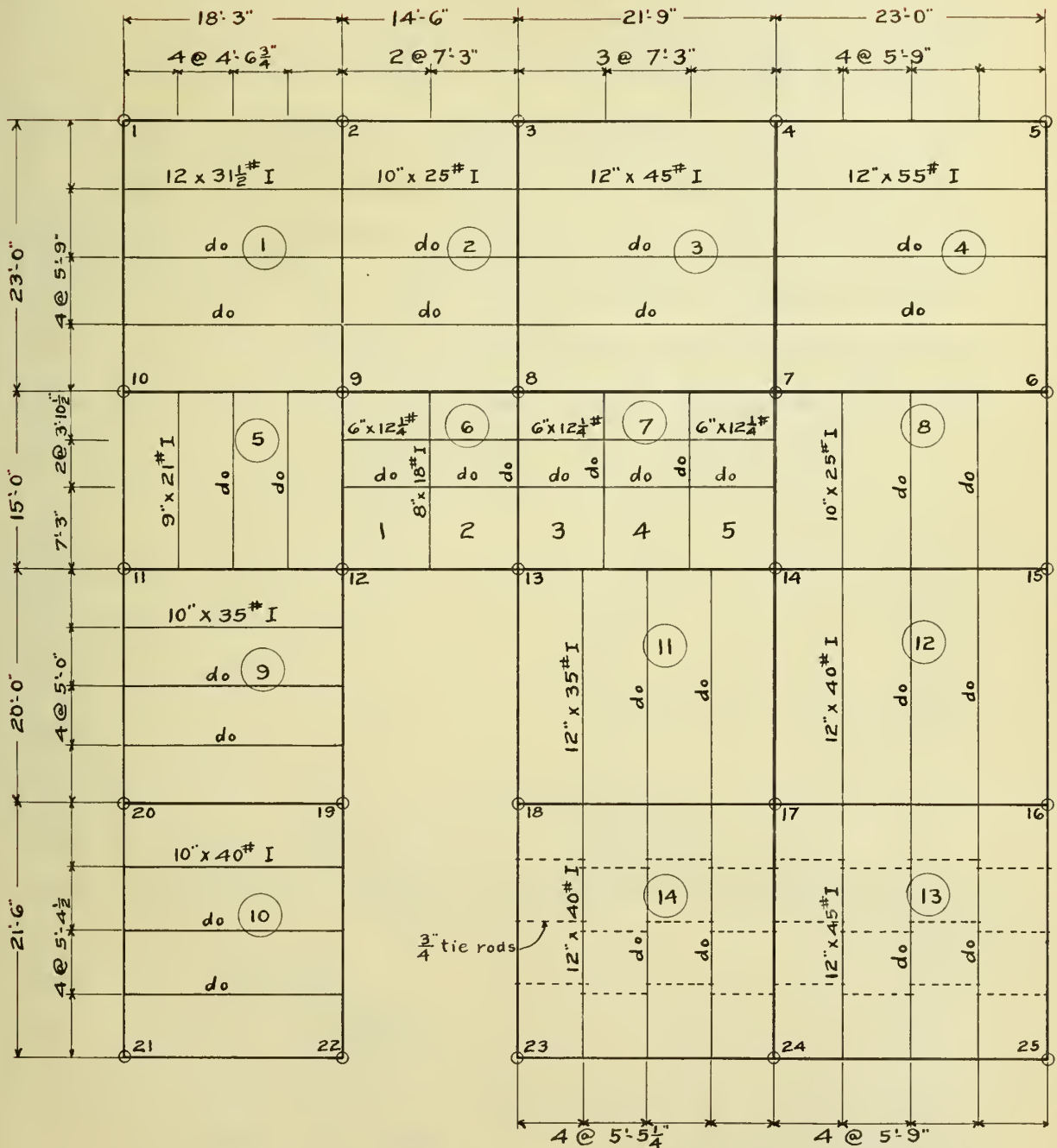
Fig. 5.

Floor Beams.

Pan- el.	Span l ft. in.	Spacing S ft.	S ft.	Uniform load. w (155xS, #	Moment. M=1/8 wl ² .	Sect- ion re- quired.	Modu- lus. Actual.	I-beam. chosen.
1	18' 3"=219"	5' 9"=5.75		155x5.75	535,000	33.4	36.0	12x31 1/2
2	14' 6"=174"	5' 9"=5.75		155x5.75	338,000	21.0	24.4	10x25
3	21' 9"=261"	5' 9"=5.75		155x5.75	758,000	47.4	47.6	12x45
4	23' 9"=276"	5' 9"=5.75		155x5.75	850,000	53.0	53.5	12x55
5	15' 0"=180"	4-6	3, 44.56	155x4.56	292,000	18.3	18.9	9x21
6&7	7' 3"= 87"3'	10	1, 23.85	155x3.85	56,800	3.55 ^x	7.3	6x12 1/4
8	15' 0"=180"	5' 9"=5.75		155x5.75	362,000	22.6	24.4	10x25
9	18' 3"=219"	5' 0"=5.00		155x5.00	465,000	29.9	29.3	10x35
10	18' 3"=219"5	4-1/2	5.375	155x5.375	500,000	31.3	31.7	10x40
11	20' 0"=240"5	5-1/4	5.438	155x5.438	606,000	37.9	38.9	12x35
12	20' 0"=240"	5' 9"=5.75		155x5.75	642,000	40.0	41.0	12x40
13	21' 0"=258"	5' 9"=5.75		155x5.75	742,999	46.3	47.6	12x45
14	21' 6"=258"5	5-1/4	5.437	155x5.438	702,000	43.8	44.8	12x40

Note-

x Excessive strength, but will be used to permit coping of top flange for clearance of terra-cotta arch.



Art. 7. DESIGN OF GIRDERS.

The girders in this building can be classified under two divisions; (1), the interior girders, which will consist of rolled I-beams; and (2), the wall supporting girders, which will be made up of riveted shapes.

(1). The interior girders are readily designed by solving for the section modulus as in the case of the floor beams. In this class there are two types of loading (see Plate XIII), as follows; Girder(7-4), which is under concentrated loads only; and girder (7-6), which is under both uniform and concentrated loads.

Design of Girder (7-4).

(Dead + 80 percent of live load) = $95 + 0.8 \times 60 = 143$ pounds per square foot.

P = end reaction of beams in panel 2 and 3 =

$$5.75 \times \frac{14.5 + 21.75}{2} \times 143 = 14,900\# \text{ Shear}$$

$$R_1 = 3P/2 = 22,400\#.$$

$$\begin{aligned} Ma &= R_1 \times 23/2 - P \times 23/4 = \\ 22,400 \times 11-1/2 - 14,900 \times 5-3/4 &= 172,000 \text{ pound feet.} \end{aligned}$$

$$I/C = \frac{172,000 \times 12}{16,000} = 129$$

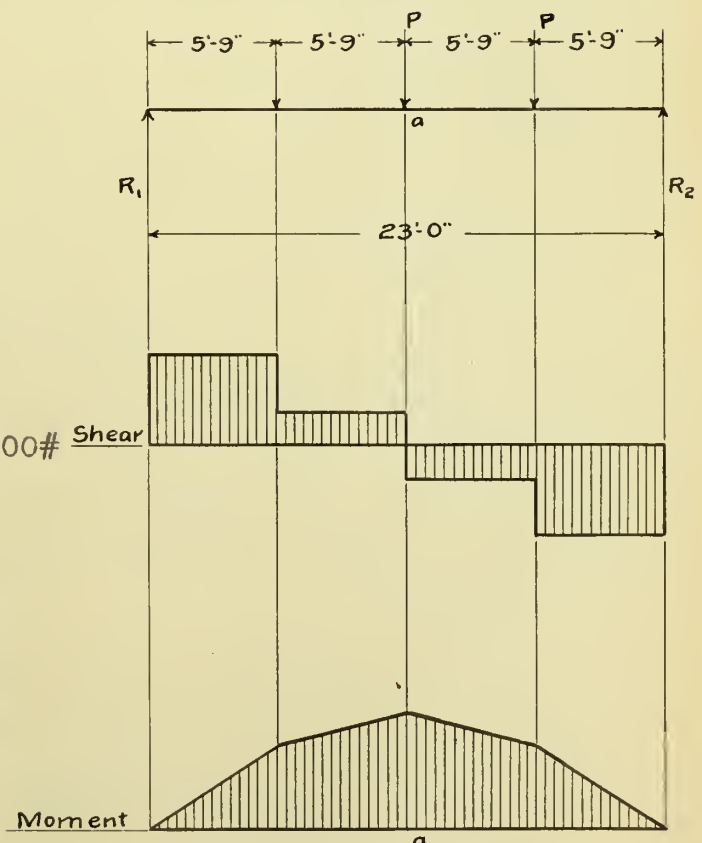


Fig. 6.

Nearest size I-beam available = 20" x 80#

Actual $I/c = 146.6$

Area of section = 23.73 square inches.

Unit end shear = $\frac{22,400}{23.73} = 945$ pounds per square inch.

Design of Girder (7-6).

Uniform load $w =$

$143 \times 5.75/2 = 6,170\#.$

$R_1 = 3P/2 + 410(23/2) =$

$9,256 + 4,720 = 13,970\#.$

$Ma = R \times 23/2 - P \times 23/4 - w \times 23/4 \times 23/4$

$= 13,970 \times 11-1/2 - 6,170 \times 5-3/4 -$

$410 \times \frac{23^2}{8} = 160,500 - 35,500 - 27,100 = 108,000$ pound feet.

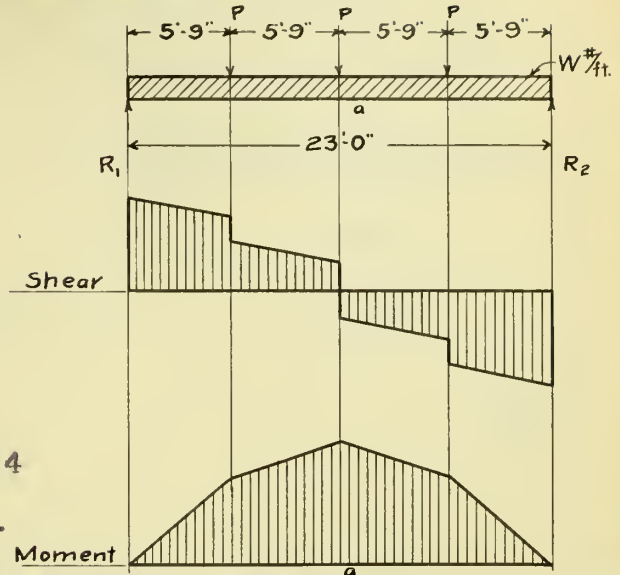


Fig. 7.

$I/c = \frac{108,000 \times 12}{16,000} = 81$

Use 18" x 55# I-beam.

Actual $I/c = 88.4.$

Area of section = 15.93 square inches.

Unit end shear = $\frac{13,970}{15.43} = 875$ pounds per square inch.

(2). The wall-supporting girders occur in two distinct forms, (a), the spandrel girders, which are simple beams, and (b), the basement cantilevers.

(a). The spandrel girders serve two purposes, in that they carry the vertical loads (floor and wall) and also serve as struts to transmit wind pressure. The first step in the design is to choose a section to carry the vertical load,

the limit of depth of the girder being somewhat less than the height of the spandrel chosen, 4 feet.

Design of Spandrel Girder (6-5), 3rd Floor.

P = floor-beam reaction. (See Plate 13).

$$= 5.75 \times 23/2 \times 143 = 9,450\#.$$

Wall area carried =

$$12.5 \times 23 - (51 + 42 + 5) = 143\#$$

$$\text{Uniform load } w = \frac{143 \times 112 \times 16/12}{23}$$

$$= 927 \text{ pounds per square feet.}$$

$$R_1 = 3P/2 + w \times 23/2 = 14,200 + 10,700$$

$$= 24,900 \text{ pounds.}$$

$$M_a = 24,900 \times 23/2 - 9,450 \times 23/4$$

$$- 927 \times 23/2 \times 23/4 = 171,400 \text{ pound feet.}$$

$$\text{Maximum end shear} = R_1 = 24,900 \text{ pounds.}$$

Try 3/8"-web plate. (Minimum allowable).

$$\text{Allowable unit shear} = 10,999 \text{ pounds per square inch.}$$

$$\text{Regular web area} = \frac{24,900}{10,000} = 2.49 \text{ square inches.}$$

Since depth of girder must exceed 1/10 of span,

try 30" x 3/8" web plate.

$$\text{Actual area} = 30 \times 3/8 = 11.2 \text{ square inches.}$$

Design of Spandrel Girder (6-5). 3rd Floor.

Bending moment is considered as resisted by flange only.

$$\text{Effective depth of girder} = 30 - 1,2 - (2 \times 2 - 1,4) = 26" = h_e.$$

$$\text{Stress in flange} = M/h_e = \frac{171,400 \times 12}{26} = 79,200 \text{ pounds.}$$

Using allowable stress of 12,000 pounds per square inch, (since member will be subject to wind stresses).

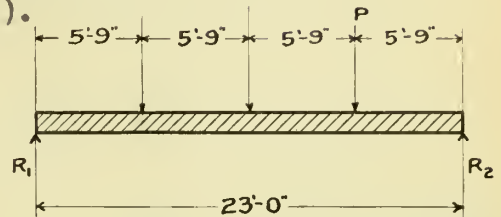


Fig. 8.

Required area of flange = $\frac{79,200}{12,000} = 6.6$ square inches.

Try 2Ls -6" x 4" x 1/2".

Actual area = $2 \times 4.75 = 9.5$ square inches.

Net area = $9.5 - 2(7/8 + 1/8) \times 1/2 = 8.5$ square inches.

This section must now be investigated for efficiency as a wind strut.

Neglecting web moment of inertia-

$$I_{yy} = 4(13.47 + 3.61 (1.94 + 1/2 \times 3/8)^2) = 118.8 \text{ inches}^4.$$

Area of section = $4 \times 3.61 +$

$30 \times 3/8 = 25.6.$

$$\text{Least } r = (I/A)^{1/2} = (118.8/25.6)^{1/2} = 2.16 \text{ inches.}$$

Allowable $l/r = 150.$

$$\text{Actual } l/r = \frac{25 \times 12}{2.16} = 128$$

$$\text{Wind stress} = \frac{15 + 12.5}{2} \times \frac{82.5}{2} \times 50 = 18,000 \text{ pounds.}$$

(See Plate VIII).

$$\text{Actual unit stress} = \frac{18,000}{25.6} = 703 \text{ pounds per square inch.}$$

$$\text{Allowable stress} = 12,000 - 70 \frac{l}{r} = 12,000 - 70 \times 128 = 3,100 \text{ pounds.}$$

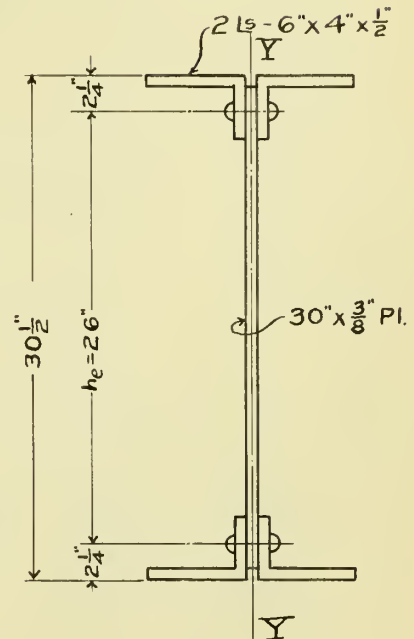


Fig. 9.

Design of Basement Cantilever. "C".

Since the radius of the supporting caisson will govern the length of the cantilever arm, it is necessary to determine the diameter of the caisson.

Total load on caisson 11 =

781,700 pounds. (See Table 5).

Allowable pressure on concrete =

10 tons per square foot.

Area required = $\frac{781,700}{10 \times 2,000} = 39$ square feet.

Diameter of caisson will be 7.5 feet, say 7 feet 4 inches.

Overhang of cantilever = $\frac{7' 4''}{2} = 3' - 8''$ to property line.

Arm of cantilever = $3' - 8'' - 1' - 6'' = 2' - 2''$.

Basement floor load on Column 11 = 48,000 pounds. (Table 2).

Load on Column 11 above basement = $781,700 - 48,000 =$

733,700 pounds.

$R_1 = \frac{733,700 \times 2.167}{10.08} = -99,500$ pounds.

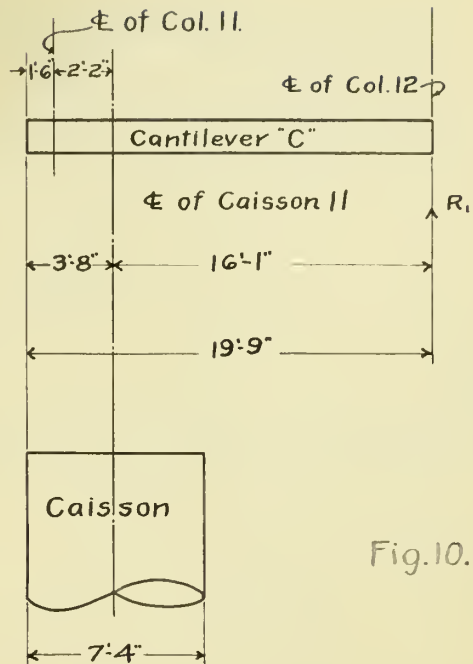


Fig. 10.

Design of Basement Cantilever "C".

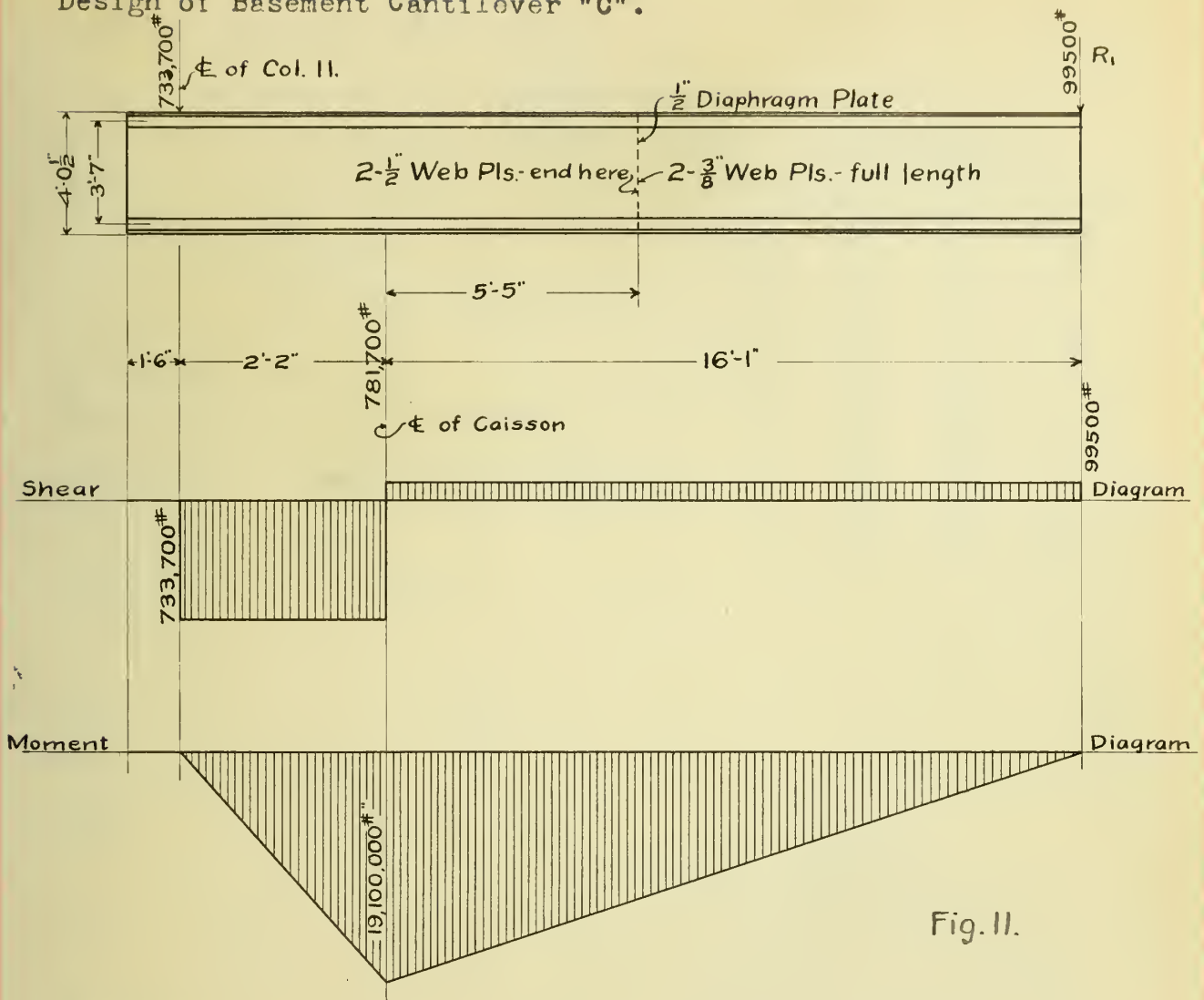


Fig. II.

Assume 48" depth of web. (Allowable shear = 10,000 pounds per square inch).

Thickness of web required = $\frac{733,700}{48 \times 10,000} = 1.53$ inches.

Try 2 Pls. -48 x 3/8, (full length of girder).

2 Pls. -48 x 1/2, (reinforcement at cantilever arm).

Total thickness = 1.75 inches.

Effective depth of girder = 4' -0 1/2" -0-5 1/2" = 3' -7" = 45 inches.

Maximum moment = 733,700 x 2' -2" = 19,100,000 pound inches.

Stress in flange = $\frac{19,100,000}{43} = 443,000$ pounds.

Required flange area = $\frac{443,000}{16,000} = 27.8$ square inches.

Basement Cantilever "C".

Try 6 Ls $-6" \times 4" \times \frac{3}{8}"$

Total area = $6 \times 3.61 =$

21.66 square inches.

Net area = $21.7 - 6(7/8 + 1/8) \frac{3}{8} =$

19.5 square inches.

Use 3 cover plates $14" \times \frac{3}{8}"$.

Net area = $42 \times \frac{3}{8} - 6(7/8 + 1/8) \frac{3}{8} =$

13.6 square inches.

Total net area of flange =

33.1 square inches.

Required net area of flange =

27.8 square inches.

Hence section assumed is ample.

Since the inner girder takes $1/3$ of total amount, it may be ended at a distance = $\frac{16' - 1"}{3}$ from the fulcrum, say

5' - 5".

To insure transmission of maximum shear from cantilever arm into main arm of the girder, the 2-1,2" web plates will be used for the full length of the inner girder = $1' - 6" + 2' - 2" + 5' - 5" = 9' - 1"$.

Before determining the exact length of this girder, it will be necessary to design Column 12 in order to get the distance from the center line of the column to the end of the girder. (See Art. 8).

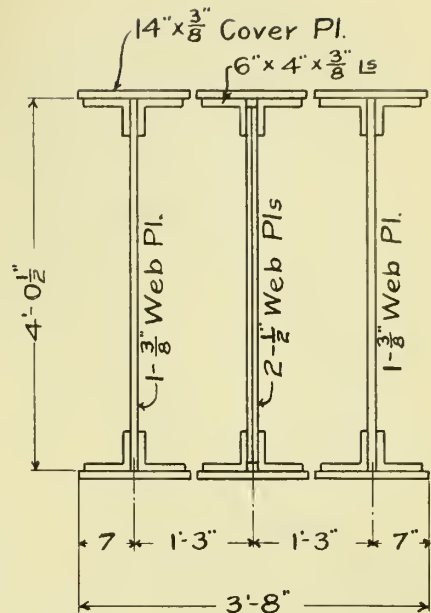


Fig. 12.

Art. 8. DESIGN OF COLUMNS.

Although precluded by the very title of this thesis, it is well to state that the use of cast-iron columns in large buildings has become a thing of the past and would no more be countenanced by progressive civil engineers than the use of cast-iron for compression members in bridges.

In choosing a type of section for a column, the designer is governed by several considerations, among which are cost of raw material, speed of shop manufacture, simplicity of girder connection, and ease of placing. There are many patented shapes on the market, which form excellent columns; but when cost of royalties and greater care in detailing the connections is considered, it is by no means surprising that the more elementary shapes are in greater use.

Buildings of this type are always designed and erected in record time; and for this reason, a column section composed of common shapes, which are usually kept in stock or are constantly rolled by the steel mills, is very desirable. In this design, the sections used will be composed of angles, channels, and plates.

Design of Column 11. (Sub-Basement).

This column is typical of a concentrically loaded compression member, of which there are but few in any building, except where used-as in this case-for a cantilever girder support.

Length of Column = $24'-4\frac{1}{2}"$.

Total load = 781,700 pounds.

Greatest unsupported length =

$13'-9" = 156"$.

Allowable stress = 16,000 -70 l, r.

Least allowable l, r = 100.

$r = l/100 = 156/100 = 1.56$

The width of this column will be determined by the size of the cantilever girder which it supports; hence the cross-

section of the column, as adopted for this support, is as shown on the next page. This is done to get a direct bearing from the channels of the column to the webs of the girders.

Minimum allowable stress = $16,000 -70 \times 100 = 9,000$ pounds per square inch.

Least required area = $\frac{781,700}{9,000} = 86.8$ square inches.

Sub-basement Column 11.

Section of Column.

Try 4[s -12" x 40#

2 cover plates -38" x 1,2".

Area of[s = $4 \times 11.76 =$

47.04 square inches.

Area of Pls. = $2 \times 38 \times$

$1/2 = 38.00$ square inches.

Total area = 85.04 square inches.

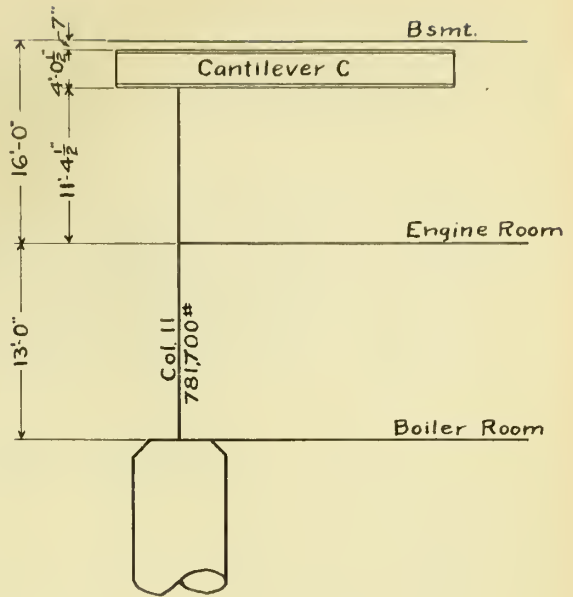


Fig. 13.

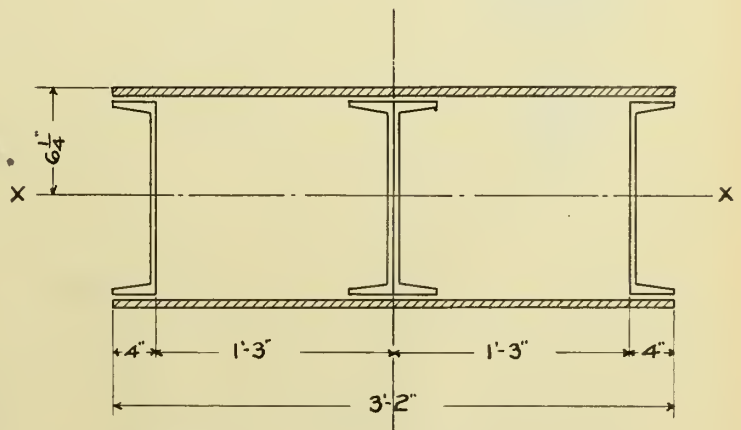


Fig. 14.

4 of
Cantilever Girder

$$I_{x_A}(\text{for } [s]) = 4 \times 196.9 = 787.6$$

$$\text{for pls} = 4(\bar{I} + Ad^2) = 4\left(\frac{38 \times (1/2)^3}{12} + 38 \times 1/2 \times (6-1/4)^2\right) = 29.696.$$

$$\text{Total } I_{xx} = 3757.2$$

$$\text{Least } r = (I/A)^{1/2} = (3757/85)^{1/2} = 6.65$$

$$\text{Allowable stress} = 16,000 - 70 (150/6.65) = 14,360 \text{ pounds per square inch.}$$

$$\text{Required area} = \frac{781,700}{14,360} = 54.3 \text{ square inches.}$$

$$\text{Actual area} = 85.04 \quad " \quad "$$

While the actual area is somewhat excessive, this section will be used for this column on account of its important function in the building. The moment of inertia about the Y axis is evidently ample to allow for wind stresses.

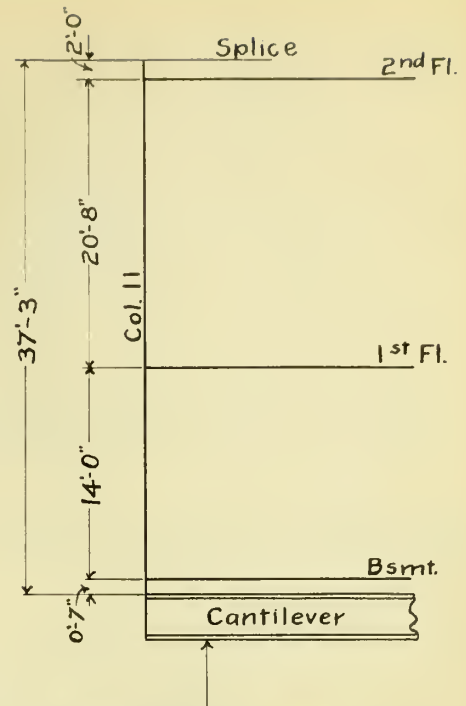
Design of Column 11- (Basement and 1st Story).

This column is chosen as a typical wall column, and will illustrate the design of an eccentrically loaded compression member. It will be spliced 2 feet above the second floor-line and will rest upon a cast steel base, in order to get the proper bearing on the girder.

While somewhat extravagant in the use of metal, the columns in a building of this type are made in two-story lengths, both for speed of manufacture and erection. Therefore, the member, being designed for the greater stress in the lower story, is not very economical when investigated for its efficiency in the story above.

The calculations for eccentricity are somewhat lengthy and no formula has yet been devised to cover all cases. Freitag (Arch. Eng. page 213) suggests the following simple method, which will here be followed.

Fig. 15.



Total load on column = 734,000 pounds.

Neglecting eccentricity, the required area = $\frac{734,000}{16,000} = 46$ sq.in.

Try 2[s -12 x 40#

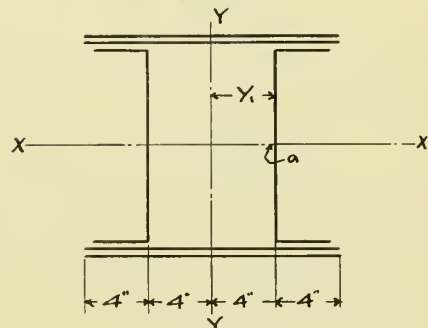
4 cover plates 16" x 1/2".

Area of [s = 2 x 11.76 = 23.52

Area of Pls = 4 x 16 x 1/2 = 32.00

Total area = 55.52

Fig. 16.



The wall loads act along the axis Y-Y and are concentric and symmetrical. The floor load from the second floor is applied at point (a) and causes the eccentric loading.

This load is equal to the floor area multiplied by the load per sq. ft. = 20,000 pounds. (See Table 2, page 30).

The stress in extreme fiber $S = \frac{My}{I - Pl^2/10E}$

Where S = stress caused by eccentricity.

M = moment of load = 20,000 x 4".

I = moment of inertia of whole section about Y-Y.

y = distance of extreme fibre from Y-Y = 16,2 = 8".

(55)

P = total load on column.

l = unsupported length of column = 20'-8" = 248".

I_{yy} for $[s = 2(6.63 + 11.76(4 + 0.12)^2)] = 536.4$

" " $P l s = 4(1.2 \times 8^3 \times 1/12) = \underline{85.4}$

Total $I_{yy} = 621.8$

$S = \frac{20,000 \times 4 \times 8}{621.8 - \frac{734,000 \times (248)^2}{10 \times 30,000,000}} = \frac{640,000}{472} = 1360$ pounds per sq. inch.

Stress due to direct load = $\frac{734,000}{55.52} = 13,200$ pounds per sq. inch.

Stress due to eccentricity = 1,360 " " " "

Actual unit stress = 14,600 pounds per square inch.

Allowable " " = 16,000 " " " "

It is here seen that eccentricity does not cause a very great increase of stress in a column of this section, and in the design of some of the largest buildings in Chicago, eccentricity has been neglected except for the use of a lower unit allowable stress, namely, 12,500 pounds per square inch. (The Fair Building).

This box-form of column section is justly regarded as one of the most desirable types both because of ease of shop manufacture and also of simplicity of splicing and connections. The section is readily increased by additional cover plates and is adaptable for a considerable range of loads without changing the outer dimensions of the box cross-section. This is a good feature when considered with regard to making the vertical splices without use of cap-plates or shims.

Art. 9. DESIGN OF WIND BRACING.

Buildings of the skeleton type cannot depend upon the walls for lateral stability, but require the steel frame itself to carry all lateral wind stresses. While the interior partitions furnish a certain degree of stiffness, this is either a very small factor, or cannot be even remotely estimated. The stiffness of the floor-arch (see Art. 3) can be relied upon to make each floor act as a rigid plane. Hence all the necessary wind bracing can be secured by using only the exterior spandrel girders as wind struts between the columns.

The simplest wind bracing, if it were possible to avoid obstructing window spaces, would be the use of the lateral rods to take up the horizontal shear between each floor. In this design, it will be seen (Plate XI) that diagonal rods can be used in the entire south face of the building, as there are ^{no} windows to obstruct. The other three exterior walls will be braced by the now prevalent method of gusset plates at the ends of the spandrel girders. The area of these plates is limited by the size of the window openings near the end of the girder.

As an example, consider the spandrel girder (6-5), third floor, which was designed in Article 7.

The depth of this girder is 30-1/2 inches, b. to b. of channels, and as it is set into the spandrel wall of 4 feet depth, it is well protected from fire.

The usual method of applying wind bracing gussets to this form of a girder is to use a 19-foot length of web plate, as shown in sketch. The flange angles will extend over the gusset plate as far as possible, the outer

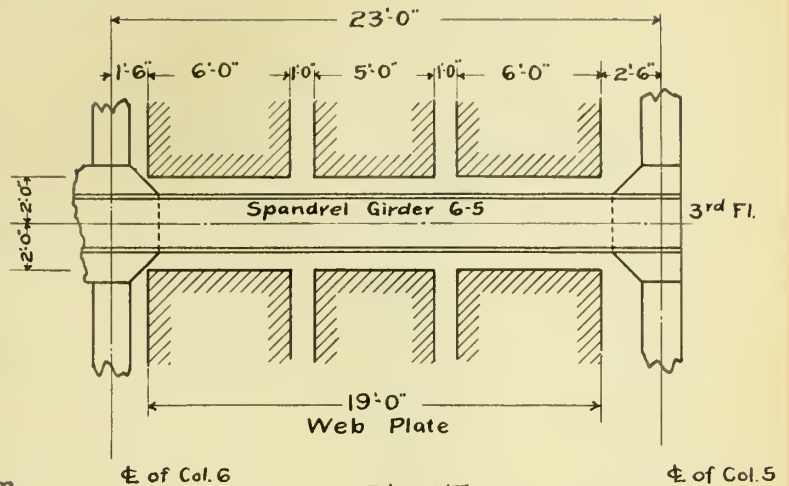


Fig. 17.

angles extending to the center line of the column and the inner angles to the edge of the column. The gusset plate will extend from the end of the web plate to the column proper and is usually cut at a bevel of from 45 to 60 degrees, according to the window clearance required.

As there are 5 tiers of columns among which the horizontal wind stress is distributed, the above bracing will be on the side of safety. Birkmire (Planning and Construction of High Office Buildings, page 204) says when the height of a building does not exceed three times its base, a well constructed frame alone is sufficient for wind bracing.

Art. 10. DESIGN OF SPANDREL-SECTION.

The spandrel-section is that part of the curtain-wall which lies between two successive column piers and between the window spaces of successive stories.

The depth of the section has been assumed as 4 feet, (see Plate IX) for all stories, and the spandrel girder is 30-1/2 inches deep. It will be desirable to have the girder frame into the column on its center line, and as the outside of the wall is 1 foot 6 inches from the center line of the steel frame, the 16 inch wall will just fit up to the web of the girder.

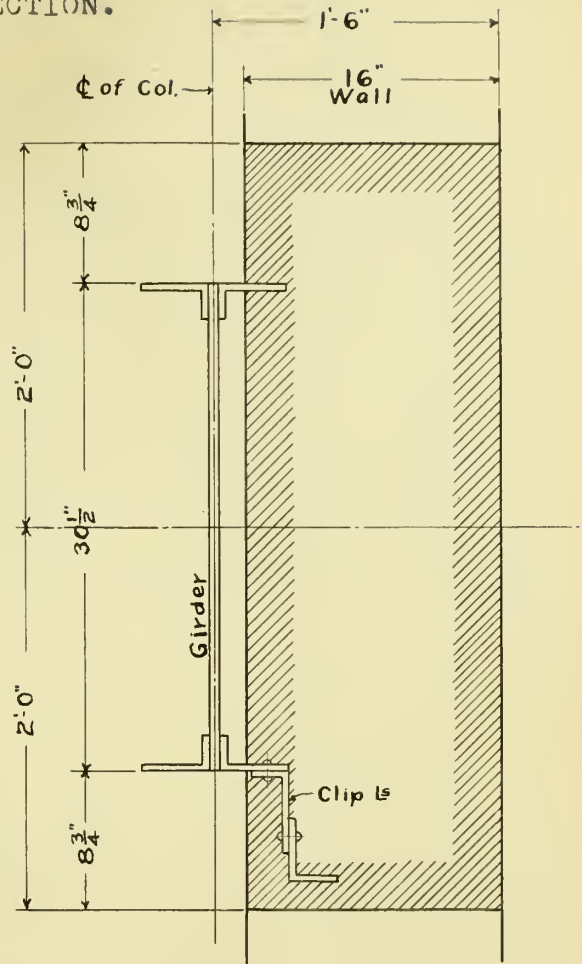


Fig. 18.

The lower portion of the spandrel is supported by clip angles which hold a single angle whose length is equal to that of the girder. When terra-cotta blocks are used they may be designed to fit the special girders and connections and then be anchored by small tie bolts.

The design of the spandrel-section will vary with the architectural treatment of the wall itself, as in some cases the spandrels are recessed or set back from the face of the piers. This results in more or less complicated bracket-work

on the spandrel girder.

The piers proper are carried directly by shelf-angles on the column itself and their weight does not, in any way, affect the girder.

The spandrel section in the light-court may be treated in the same manner. Here the wall is faced with white tile for the better diffusion of light. As there is no wind bracing needed on these spandrel girders and as the wall is only 12" thick, it will be found convenient to frame the girders into the columns about 6 or 8 inches outside of the center line of the frame, thus bringing the girder nearer the center line of the load which it is to support.

CONCLUSION.

This design has merely considered the more general features of a modern steel building. In any specific case, localized loads would have to be reckoned with, such as safes and vaults. But the modern trend is toward the use of the equivalent uniform load-system; and in a building so designed, partitions can be altered to suit any future demand.

While there has never been a failure of a building of this type, it will be found advisable to subject every feature of a design to the most rigid analysis possible and not merely trust to past performances. It is true that a steel building of this type cannot collapse as a concrete building would, but its failure would be a severe blow to the reputation of the American engineers who originated it.

When properly designed and with all vital members protected from fire and corrosion, such a building is as enduring as the geological formation upon which it rests, and the only limit to its life is the changing demands of business which may demand its removal to provide room for a larger structure.

THE END.





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